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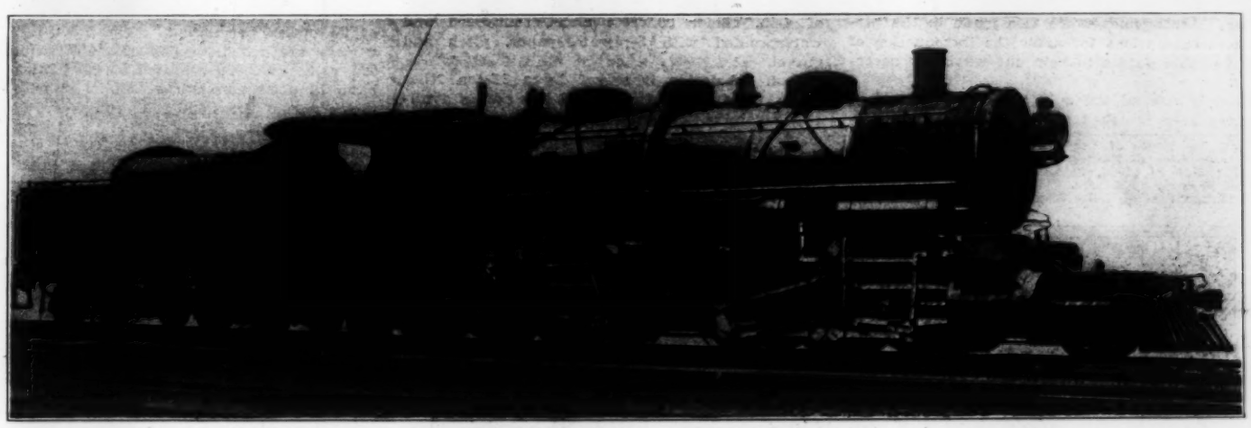
MALLET DUPLEX COMPOUND FREIGHT LOCOMOTIVES FOR THE GREAT NORTHERN RY.

The great duplex locomotive of the Mallet system which was built about two years ago for the Baltimore & Ohio Ry. (Eng. News, June 30, 1904, and June 15, 1905), and which was one of the notable exhibits at the St. Louis Exhibition, has now been followed by some very similar but slightly larger engines for the Great Northern Ry. These latter have recently been completed by the Baldwin Locomotive Works, and one of them is shown by the accompanying illustration. The Baltimore & Ohio Ry. engine was designed for pusher service on heavy mountain grades, and weighs 334,000 lbs., the entire weight being carried on six driving axles arranged in two groups; the three rear axles are carried in the main frames, and the three front axles are carried in a swiveling truck.



COMPOUND DUPLEX LOCOMOTIVE ON THE MALLET SYSTEM FOR THE GREAT NORTHERN RY.
Baldwin Locomotive Works, Philadelphia, Pa.; Builders.

The new engines of the Great Northern Ry. are intended for use as regular road engines in heavy freight service, and have at each end a pony or two-wheel truck in order to facilitate the passage of the engines around curves of 10°. The total weight is 355,000 lbs., being slightly greater than that of the earlier engine, but the weight on driving wheels is somewhat less, 316,000 lbs. The earlier engine is of the 0-6-6-0 type, while the later engines are of the 2-6-6-2 type.

The boiler of the Great Northern engine is about the same in size as that of the B. & O. Mallet. With a firebox heating surface of 225 sq. ft. (against 219), and a tube heating surface of 5,433 sq. ft. (against 5,366), the Great Northern locomotive has a steaming capacity about 2% greater than its predecessor. The grate area, 78 sq. ft. (against 72), is increased in somewhat larger ratio. The later locomotive is, however, designed to carry a lower boiler pressure than the other (200 lbs. per sq. in. against 235). In discussions on the B. & O. locomotive the opinion has been repeatedly expressed that steam pressures in excess of 200 lbs. are of doubtful advisability, on account of the greater liability to leakage and greater cost of maintenance which they entail. The more conservative pressure of the Great Northern engine no doubt is a concrete expression of this view. The lower pressure is in part made up by larger cylinders; the newer engine has 21.5-in. h. p. and 33-in. l. p. cylinders (against 20 and 32 in the B. & O.), with stroke the same as in the earlier engine (32 ins.).

The boiler and firebox of the Great Northern engine are carried well above the frames. The latter are of cast-steel. The rear frame, which carries the high-pressure cylinders, is in rigid connection with the boiler, while the forward frame, with the low-pressure engine, swivels around a vertical pin in the center line of the saddle of the rear engine. A rearward extending radial frame bolted to the top of the forward engine frame, projects back to engage with this pivot. The boiler has a bearing near midlength of the forward frame, where a heavy cross-girt of the frame provides a seat for a tranverse sliding bearing. The receiver pipe, which carries the exhaust of the high-pressure engine forward to the low-pressure cylinders, swivels at the same point as the forward frame, and is provided with a slip joint. The exhaust from the low-pressure (forward) cylinders calls for another flexible pipe connection; the exhaust stand is rigidly fixed in the smokebox, and the short connection pipe from the top of the low-pressure cylinder casting is fitted with swiveling and expansion joints to accommodate itself to the lateral motion of the forward frame relative to the boiler. The two pony trucks are pivoted by radial frames in the usual way.

The firebox is of the Belpaire type (for soft coal), with a very slightly curved crown sheet and flat slides flaring outward from the crown sheet to the mud ring. There are two fire-doors. The dome is a steel casting, and being very low it is fitted with a throttle valve of the Rushton pattern, having side openings. The main steam pipes are carried from each side of the dome to the high-pressure cylinders in front of the rear group of driving wheels. The high-pressure exhaust is delivered to a receiver pipe extending forward between the frames and forked to reach the valve chests of the low-pressure cylinders in front of the leading group of driving wheels (of steam trucks); this pipe is of flexible construction as already noted. Balanced slide-valves are used for both high and low-pressure cylinders. They are operated by valve gear of the Walschaert type. The reversing gear is operated by a compressed-air device.

The general dimensions of these huge engines are as follows, being given in our standard form:

Driving wheels (12, in two groups)	4 ft. 7 ins.
Truck wheels (4, in two trucks)	2 ft. 6 ins.
Journals, driving axles	10 × 12 ins.
Wheelbase, driving (each group)	10 ft. 0 ins.
Driving (both groups)	30 ft. 0 ins.
Total engine	44 ft. 10 ins.
Engine and tender	73 ft. 2¼ ins.
Weight: On driving wheels	316,000 lbs.
On front truck	19,000 lbs.
On rear truck	20,000 lbs.
Engine, total	355,000 lbs.
Engine and tender	503,000 lbs.
Cylinders, h. p. (2)	21½ × 32 ins.
Cylinders, l. p. (2)	33 × 32 ins.
Valve gear	Walschaert
Slide valves	Balanced
Boiler, type	Straight-top
Diameter	7 ft. 0 ins.
Barrel plates	⅞-in.
Circumferential seams	Triple and double riveted
Working steam pressure	200 lbs.
Firebox, Belpaire; size	9 ft. 0 ins. × 8 ft. 0 ins.
Depth, at front, 6 ft. 7⅛ ins.; at back	6 ft. 4½ ins.
Crown, side and back plates	⅜-in.
Tube sheet	½-in.
Crown stays, Vertical; diameter	1⅛ ins.
Water spaces; front, 6 ins.; sides and back	5 ins.
Tubes; No., 441; thickness	No. 11, W. G.
Spacing	2¼-in.
Diameter, 2¼ ins.; length	21 ft. 0 ins.
Heating surface; tubes	5,433 sq. ft.
Firebox	225 sq. ft.
Total	5,658 sq. ft.
Grate area	78 sq. ft.
Water in tender tank	8,000 gallons
Coal in tender	13 tons
Tractive power (working compound)	71,800 lbs.

THE ROTARY CONVERTER SUBSTATIONS AND ELECTRIC CAR EQUIPMENT OF THE LONG ISLAND RAILROAD.

The third-rail electric service of the Long Island Railroad includes both local and express trains consisting of two to six cars supplied with 600-volt direct current. Both permanent and portable substations are used, the latter being noteworthy as an innovation in electrical distribution systems. Another special feature is the use of an adjustable third-rail shoe for the adaptation of the Brooklyn Rapid Transit cars to the third rail of the Long Island Railroad, which latter has a lower and more extended third rail than that portion of the Brooklyn Rapid Transit Co.'s lines so equipped. These and other features of the substations and car equipment are described and illustrated in some detail herewith.* In the preparation of the present article we have drawn freely upon the material collected by Mr. W. N. Smith, of Westinghouse, Church, Kerr & Co., who has been closely associated with the work described.

Five permanent substation sites were selected, their locations being the most advantageous points from which to distribute 600-volt current to the cars under the attending traffic conditions. Owing to the heavy loads incidental to race track traffic, which occur on the Long Island Railroad for only four weeks during the entire year, at two points on the Long Island suburban lines, it was considered best to employ portable substations, instead of permanent ones, for supplying the extra current at these points. Each of the two portable substations consists of a 1,000-KW. rotary converter with three 375-KW. transformers and a switchboard, all mounted in a steel car. As only one race track is in operation at a time, both of these substation cars are operated together, first at one track and then at the other. Arrangements are also made in the permanent substations to enable the introduction of the portable outfits to help out whenever necessary.

In only one of the substations has a storage battery been installed as an adjunct to the machinery, although the location and arrangement of all the substation buildings are such as to enable the ultimate use of a storage battery should future conditions justify it. The battery installed consists of 300 cells having a capacity of 3,200 ampere-hours at the one-hour rate, with 600 volts. With it two boosters of 162-KW. capacity each are used. This equipment is placed in the substation farthest from the power station, at a point where the load is heavy in the summer and very light in winter.

*See also issues of Engineering News: Nov. 2, 1905, The Installation of Electric Traction on the Long Island R. R.; May 31, 1906, Structural Details of the Long Island Power Station; June 14, 1906, Transmission and Distribution System, Long Island R. R.

SUBSTATION BUILDINGS.

The space inside the substation buildings is taken up by a central section of about 51-ft. span, which is wide enough to accommodate two 1,500-KW. rotaries and the necessary transformers. This central space is flanked on each side by a narrow section, that on one side being devoted to high-tension bus bars and switch gear, and that on the other side to the low-tension switchboard and operating gallery. The buildings vary in height according to the requirements of the overhead high-tension circuits. Usually, in the main portion of the building, the height is about 30 ft. from the main floor to the bottom of the roof trusses. Each story of the side galleries is about 12 ft. in height. Where only underground circuits are used, there are but two floors in the high-tension gallery. Where overhead circuits are employed, a third floor is added; this third floor serves the double purpose of affording a convenient entrance to the building for the cables, about on a level with the cross-arms, and at the same time provides a suitable chamber for enclosing the high-tension lightning arrester apparatus.

The substation walls are entirely of brick, with the window sills, cornice, coping and name panel of terra cotta. The floors throughout are of reinforced concrete slabs, and the roofs are of 4-in. cinder concrete slabs, reinforced and waterproofed with 5-ply tar-paper, pitch and gravel.

Each substation is served by a 16-ton Niles crane, which spans the central bay and is available not only for handling the permanent heavy machinery but also the apparatus of the portable substations, which latter can be run under the crane upon a track that enters one of the end bays of the building. The crane is hand-operated from its trolley, thus avoiding the usual hanging chains.

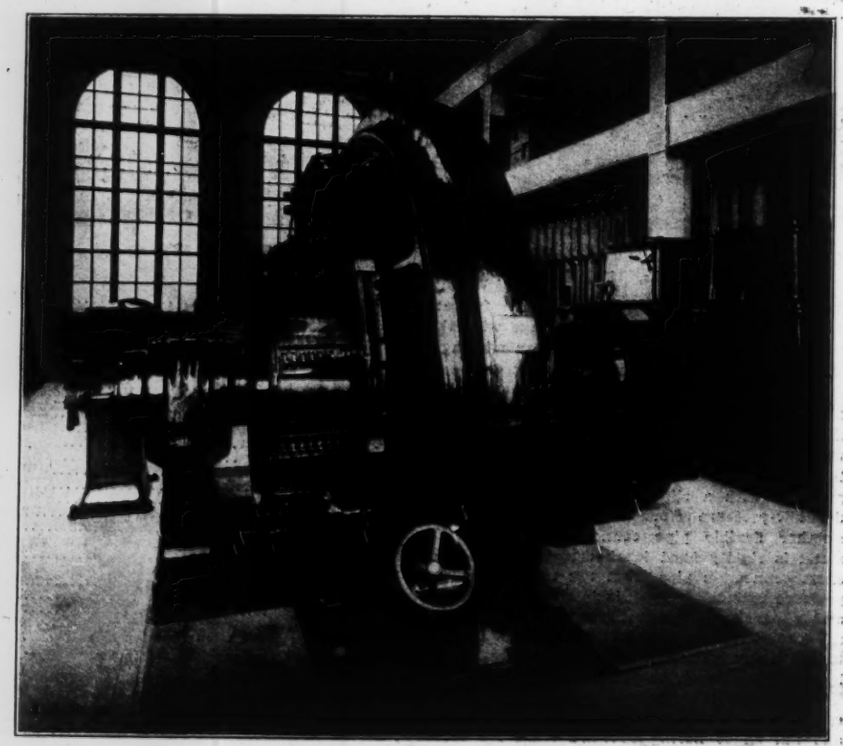


FIG. 1. VIEW OF 1,500-KW. ROTARY CONVERTER IN ONE OF THE SUBSTATIONS OF LONG ISLAND R. R., SHOWING ALSO THE GALLERY CONTAINING THE HIGH-TENSION OIL CIRCUIT BREAKERS AND BUS BARS.

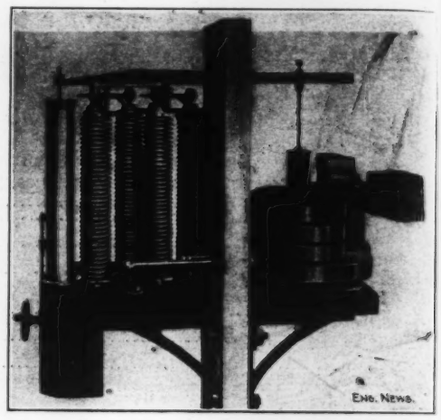


Fig. 2. Carbon Regulator for Storage Battery.

ROTARY CONVERTERS.

The foundations upon which the rotary converters rest are of concrete, and upon these the machines are placed without anchor bolts. The converters, of Westinghouse manufacture, are each provided with a starting motor whose frame is mounted upon an extension of the base of the rotary converter. Two sizes of converters are used, the capacities being 1,000 KW. and 1,500 KW. The 1,000-KW. machines are rated to give 1,600 amperes at 625 volts, and 1,667 amperes at 600 volts. The three-phase potential at the alternating end is about 370 volts for 625 volts at the direct-current end. The 1,000-KW. machines have eight poles and operate at 375 r. p. m., corresponding to a frequency of 25 cycles per second. The 1,500-KW. converters are rated to deliver 2,400 amperes at 625 volts, or 2,500 amperes at 600 volts. They have 12 poles and run at 250 r. p. m. Fig. 1 shows one of the 1,500-KW. converters installed in a substation.

TRANSFORMERS.

Air blast transformers are used, and air is forced into them by blowers connecting with ducts terminating in the main floor beneath the transformers. The transformers for the 1,000-KW. rotary converters are grouped in banks of three 375-KW. units to one converter. For the 1,500-KW. converters they are in groups of three 550-KW. units each. The shell type of construction is used, and the high-tension winding is designed for a normal E. M. F. of 12,000 volts with taps arranged to enable other voltages to be utilized down to 10,000 volts. The low-tension winding is designed to normally carry 400 volts. By means of taps, other voltages may be taken off it down to 340 volts.

The 375-KW. transformer requires about 15,000 cu. ft. of free air per minute, and the 550-KW. transformer 2,000 cu. ft. per minute with a pressure of 1 oz. per sq. in.

In each substation there are, besides the main transformers, four sets of auxiliary transformers. One set supplies current to the rotary converter starting motors; another set furnishes current to the motors driving the booster generators and their exciters; a third set is used for driving the transformer blower motors and an induction motor-generator set used to charge the small auxiliary storage battery that supplies current for the electric switch control system; and a fourth set supplies current for the substation lighting.

STORAGE BATTERY EQUIPMENT.

Storage batteries, as previously mentioned, are provided at one substation only. This equipment comprises 300 elements of "Chloride" accumulator made by The Electric Storage Battery Co., of Philadelphia, Pa. Each element contains 35 plates. At the normal 1-hr. rating, the battery will discharge at the rate of 3,200 amperes. For instantaneous fluctuations, however, it can discharge up to a momentary maximum rate of 9,600 amperes per hour.

The boosters used in connection with the battery are separately excited, and each of them consists of one three-phase induction motor and one direct-current generator mounted on a common bed-plate. The booster generator gives 1,200 amperes at 135 volts continuously. The booster motor is designed for 25-cycle, three-phase current at 400 volts. It is rated at 235 HP.

The two boosters in parallel can completely discharge the battery in one hour. They are connected in series with the battery and the third rail, and in order for the battery to maintain a comparatively steady load on the rotary converters, as is desirable, the boosters must have their field excitation changed both as to strength and polarity by an amount that will cause the boosters to either charge the battery from the converters or when reversed to discharge it into the third-rail circuit. This regulation is effected almost instantaneously with the fluctuations as they occur in the system, by a carbon regulator made by the Electric Storage Battery Co. and shown in Fig. 2. Its action is somewhat analogous to that of the Wheatstone bridge, and by means of it the load upon the converters can be prevented from fluctuating by more than 5% on either side of the average.

HIGH-TENSION BUS BARS AND CIRCUIT BREAKERS.

At the substations the incoming cables of each circuit runs first to an oil switch and thence up to the transfer bus in the gallery overhead, each cable being run in a separate brick duct. The outgoing branch circuits are tapped out of the transfer bus, each tap coming down a duct in the back wall of the bus structure to the oil switch directly underneath. The oil switches, both for the incoming and outgoing circuits, are all in the same line underneath the transfer bus on the outer side of the gallery. On the inner side of the gallery, the rotary buses and the switches connecting them to the transformers and rotary converters in the station are arranged in line in like manner.

Between every oil switch and the bus to which it is connected is a set of three disconnecting knife switches, one in each phase. They are opened and closed by hand by means of a hook on the end of an insulating wooden rod. The blade of the switch is hinged on the dead side of the connection when open. These switches serve to isolate the circuit breakers from the bus bars to facilitate inspection, cleaning and repairs.

DIRECT-CURRENT SWITCHBOARDS.

Direct current for the third rail and track circuits is distributed from the rotary converters to the outgoing circuits through marble panel switchboards of standard direct-current type. These switchboards are situated in the operating gallery, Fig. 3, which also contains the high-tension control apparatus and is directly opposite the gallery (shown at the right of Fig. 1) containing the high-tension oil circuit breakers and bus bar connections. All the essential switching and control apparatus used in regular operation is, therefore, easily handled by a single operator. The only exceptions to this are the equalizer and negative rotary converter switches which are on pedestals adjacent to the rotary converters on the main floor.

FIG. 3. OPERATING GALLERY CONTAINING THE ALTERNATING-CURRENT CONTROL APPARATUS AND THE DIRECT-CURRENT DISTRIBUTING APPARATUS.

PORTABLE SUBSTATIONS.

The 1,000-KW. rotary converter and the 375-KW. air-blast transformers used on the cars for the two portable substations are identical with those used in the permanent substations. High-tension current is taken into the portable substation through three connecting lugs which afford a connection to three flexible leads supported over the track in the fixed sheds for the portable substations. Direct current is led out of connecting lugs placed opposite an opening in the side of the car. Lightning protective apparatus for the high-tension circuits is located permanently in the tower at one end of the building. Permanent low-tension connections mounted on the wall within the substation building are joined directly to the third rail and the tracks.

The portable substation converters are synchronized from the low-tension side by means of a synchroscope. The transformers are kept cool by a 50-in. steel fan which delivers 4,500 cu. ft. of air per minute at 1 oz. pressure when running at 710 r. p. m. This fan is driven by one 3-HP. three-phase, 400-volt induction motor receiving current from the alternating-current rotary panel.

CABLES AND WIRING.

The incoming and outgoing high-tension feeder cables are of 250,000 c. m. cross section. Feeders entering a station by way of the overhead high-tension gallery are of bare copper as far as the choke coils. Feeders entering from the underground conduits are of 3-conductor cable, each conductor insulated with 7/32-in. of paper, and the group of three conductors covered with jute. Another layer of 7/32-in. paper encloses the entire group, and the whole is incased in a lead sheath 9/32-in. thick. The direct current third-rail feeders are 2,000,000-c. m. cables. They leave the building through a conduit system which leads them to their respective destinations at the third rail in front of the substation. The direct-current leads to the portable substation connections are two 1,500,000-c. m. cables.

HIGH-TENSION LIGHTNING ARRESTERS.

Westinghouse lightning arresters are used. They are mounted on marble panels which are carried on a framework of steel angles. The three arresters on the three legs of the high-tension circuit are separated by barriers of asbestos lumber. There is a choke coil in series with each main circuit, mounted near the top of the steel framework.

HEATING AND LIGHTING OF SUBSTATIONS.

Each substation is fitted with a hot water heating system supplied from a boiler in the basement. The radiators are mostly of the two-column type, averaging about 150 sq. ft. of heating surface.

The substation lighting current is taken from one of the auxiliary transformers which supplies a 105-volt three-wire lighting bus. The five substations have an average of 160 lamps each; of these, 153 are incandescents and 7 are Nernst lamps. The incandescents are 16 c. p., and the Nernst lamps have four glowers each.

TRANSFORMER BLOWER OUTFIT.

Electric-driven blowers are used for furnishing the necessary air-blast to the transformers. Two sizes are employed: one operated by 9.8-H.P

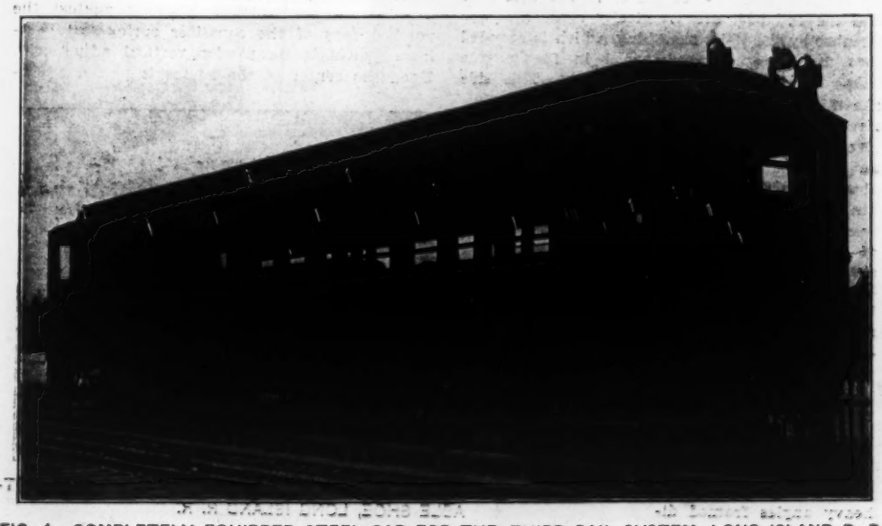


FIG. 4. COMPLETELY EQUIPPED STEEL CAR FOR THE THIRD RAIL SYSTEM, LONG ISLAND R. R.

FIG. 5. ADJUSTABLE THIRD-RAIL SHOE, INNER POSITION.

Fig. 6. Adjustable Third-Rail Shoe, Outer Position.

motors, and the other by 6.6-HP. motors. The larger size runs at 480 r. p. m. and delivers 18,000 cu. ft. of air per minute against a pressure of 1 oz. per sq. in., and the smaller size operates at the same speed and delivers 12,000 cu. ft. against the same pressure. The motors are coupled direct to the fans, and are of the three-phase induction type operating at 400 volts from the auxiliary transformers.

SIGNAL SERVICE APPARATUS.

The railroad lines are protected by a block signal system, operated by alternating current; the two rails of the track form the signaling circuit. The signaling current is obtained from a set of transformers in one of the substations. There are two transformers of 100 KW. each for this purpose; one only is used, the other being a reserve unit. These take 11,000-volt, single-phase current and transform it to 2,200 volts for the signal system.

AIR CLEANING SETS.

At each substation an air compressor, driven by a 600-volt, direct-current motor, supplies air to a system of piping running to outlets conveniently placed for cleaning the rotary converters and switching apparatus. The capacity of these cleaning sets is 50 cu. ft. of air per min., up to 100 lbs. pressure.

TRAIN SERVICE.

The traffic conditions necessitate express trains of three cars each and local trains of two or three cars, except during the rush hours, when six cars are used on the locals. The express trains average about 28 mi. per hr. including stops, and the local trains about 22 mi. per hr. including stops. The success of the steel passenger cars on the New York Subway induced Mr. George Gibbs, who designed them and who later became chief engineer of the Long Island Railroad electric equipment, to advocate their use on this road as well. To the Interborough Rapid Transit Co. and the Long Island Railroad Co. belongs, therefore, the distinction of being the first railways in the world to adopt this radical departure in car construction.

As the Long Island Railroad cars were obliged to meet very similar conditions to those in the New York Subway, the cars were built along practically the same lines as the all-steel passenger cars of that road. The one exception is the steps which are made necessary by the low platforms at stations in the suburban districts. As the multiple-unit system of control was adopted, allowing the distribution of motive power under all the cars, there was no need to keep down the weight of the individual units, and their parts were built with a view to greater rigidity and ability to resist shocks than is usual in the case of steam railway coaches. The steel construction did not, however, materially increase the weight over what it would be with durable construction in a wooden car, and the advantages gained were decreased fire risk, greater strength and increased durability.

CAR BODIES.

The center sills of the car body framing consist of two heavy I-beams which are continuous between the platform end sills. The side sills of heavy angle iron, and two extra iron sills extending back to the body bolster support the vestibule platforms. The bolsters are of the built-up type, the top and bottom members being made of two rolled steel plates. Between these plates and between the center sills is placed a malleable iron draw casting, machined so as to be a perfect fit between the bolster plates and the longitudinal sills to which it is bolted. Between the center and side sills, malleable iron struts are inserted. The body center plates are of cast steel, machined to fit the truck center plates and the bottom member of the body bolster.

In the floor framing the belt rail, side sills and side posts are riveted together at their intersections, making a truss of square panels so reinforced by the side sheathing of the car that they constitute a stiff truss without the necessity of diagonal bracing. Four sets of diagonal braces reach down from the side posts of the car below the belt rail and are fastened to the bottom framing. Both platform and sills are of heavy angles, bent to give the proper shape, and are continuous from side to side. The vestibule end posts are fastened to them with heavy steel castings.

Each of the body corner posts consists of an angle bar fastened to the end sill and the end plate, which is reinforced by a Z-bar, and also by the first side post of the car. The first post and the end post are practically made into one member by the ⅛-in. sheathing, which is rounded into the form commonly used in the outside shape of the corner post of a car. This side sheathing is securely riveted by round-head rivets to the post, making an end post construction about 3 ins. thick and 10 ins. long. There are sixteen posts along the side of the car, of which six are made of two 3 × 2 × ¼-in. angles, spaced 6 ins. apart. The remaining side posts are single 3 × 3 × ¼-in. angles.

To the top of the side posts are secured side plates of 4½ × 3-in. angles, which are in one piece from end to end of the car vestibule hoods. The end plates are of heavy angles framed directly across at about 4 ins. above the level of the side plates, and are connected to the latter by a pair of heavy angles forming a sort of Z-connection. This enables the end plates to be raised high enough to admit of the insertion of the steel door rails at the proper height for carrying the sliding doors in the ends of the car body, and also facilitates the curving of the roof.

The roof framing of the car bodies consists of bent steel angles, reinforced at the bends. A very light roof covering of composite board is used (except that sheet steel is employed over the vestibule) the whole being covered with heavy canvas laid on with white lead. The vestibule is fitted with side doors arranged to slide in pockets in the sides of the car, leaving the entire platform to the passengers. The end door when in the extreme open position is folded over the master controller, brake valve and all other apparatus in the motorman's compartment. When this door and the side doors are closed, the entire vestibule is available as a motorman's compartment, and the control apparatus is entirely exposed. The device for operating the side doors consists of a series of bell cranks and levers so arranged that the movable parts are either overhead the vestibule or entirely outside of it, leaving the entire interior of the vestibule clear. In the vestibule at the motor end of the car, just forward of the end door pocket, is placed a swinging door of pressed steel, made convex to hold the auxiliary control switchboard panel. There is also an air whistle provided over each vestibule, with its operating valve near to the control apparatus. Wooden paddles are carried in the vestibules of all cars, which are used to raise the contact shoes from the conductor rail when necessary.

The sheathing of the car consists of steel plates ⅛-in. thick, the joints between the plates being covered by means of steel battens laid on with thick red lead and secured by rivets. The flooring is of corrugated sheet iron supported by longitudinal sills and steel-plate bridging that is riveted across the space between the sills. The corrugated sheets are provided with metal clips for holding a "Monolith" plastic floor, upon which, after being finished, maple floor strips are laid. The Monolithic flooring is absolutely fireproof and is laid on in the form of a cement.

Fig. 4 gives a view of the completely equipped steel car, showing the vestibule end door closed, as it appears when the vestibule is being used as the motorman's compartment. There are 130 motor cars and four trailers, all of which were built by the American Car & Foundry Co., at Berwick, Pa. They were designed, however, by Mr. George Gibbs, as previously stated.

TRUCKS.

The wheel base of the motor trucks is 6 ft. 8 ins. for 36-in. wheels, and for the trailer trucks 5 ft. 6 ins. for 30-in. wheels. The distance between truck centers is 34 ft. The third-rail shoes are of the hinged slipper type supported on a wooden beam which is clamped against the notched face of the equalizer spring-seat castings, providing means for vertical adjustment. Upon the center of the beams is placed a 600-

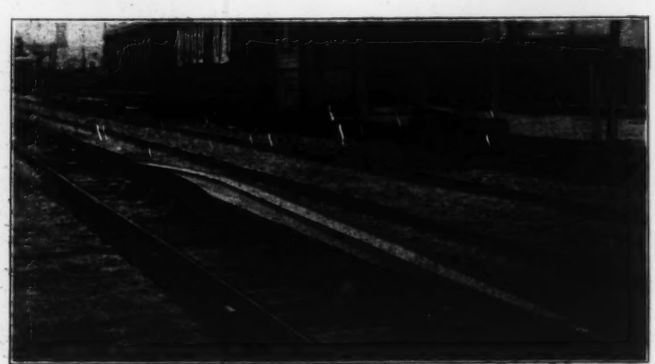


FIG. 7. CAM IN THIRD RAIL FOR CHANGING POSITION OF THE ADJUSTABLE SHOE, LONG ISLAND R. R.

amp. enclosed fuse in a wooden asbestos-lined box with hinged cover.

The Brooklyn elevated lines have been for some years operated by the third rail, but the location of their rail is 22¼ ins. outside and 6 ins. above the track rail, while the Long Island Railroad third rail is 26 ins. out and 3½ ins. up. As it was desirable to have the Brooklyn elevated trains operated over parts of the Long Island lines, an adjustable third-rail shoe was fitted to those cars of the Brooklyn Rapid Transit Co. thus used which would operate with equal facility over both third rails and be able to change from one to the other at reduced speed without requiring attention on the part of the motorman or train crew. The arrangement used was worked out, and patents on it have been applied for, by Mr. James C. Boyd. It consists essentially of a hinged slipper type of shoe mounted upon a movable lug which is held in either position by means of coil springs and is actuated by an arm that engages with a stationary cam mounted alongside the track, in line with the third rail. The movement of the car past this cam in one direction changes the shoe from the inner to the outer low position, and a reverse movement of the car past the cam changes it from the outer to the inner raised position. Fig. 5 shows the shoe in the inner position; Fig. 6 shows it in the outer position, and Fig. 7 illustrates the cam as it appears in the third rail.

BRAKES.

The cars are equipped with hand brakes and Westinghouse air brakes. Compressed air is supplied to the system by a Westinghouse electrically driven air compressor on each motor car. The compressor has a capacity of 24 cu. ft. of air per min., and supplies both the air brake and the pneumatic control system. The brake and control systems, however, are operated from separate reservoirs.

MOTORS.

Careful investigation showed that the greatest flexibility and simplicity, and a minimum first cost and maintenance would result from a two-motor car equipment, using the most powerful motors practicable. The dimensions of the largest trucks that could be operated under the conditions prescribed by the tunnel and curve clearances restricted the size of the motor to about 200 HP. The motors are of the Westinghouse type and both are mounted on one truck, a nose suspension being used.

CONTROL SYSTEM.

The Westinghouse electro-pneumatic multiple-unit control system was adopted. Its special features are as follows: Air operation of the main switches; use of storage battery current for control of the main switches; indirect system of control from a small master controller with small currents and train line conductors; automatic progression of switch operation in starting a train, and the protective features.

The advantage of air pressure as an actuating force for making and breaking switch contacts is that it permits an application of considerable power at the contact with relatively light and simple means, consisting simply of a piston working in an air cylinder, making contact by air pressure and breaking it by a powerful release spring when the air is exhausted. The use of storage-battery current at 14 volts pressure for controlling the main switches removes the necessity for using line current at 600 volts and thus eliminates any bad effects that might result from a fluctuation of the potential of the system, and the insulation of the apparatus is far easier to maintain in working order. In the indirect system of control there are main switches on each car actuated by the movement of a master controller by the motorman. This dispenses with the concentration of a large number of cables and contacts where arcs are likely to be frequent and heavy. The automatic feature of operation is important in securing a regular progressive action of the switches independently of the manner in which the motorman may handle the controller, or of any accident that may happen to the train line.

The control system may be divided into two general sections, as follows: The main motor control system through whose connections the current from the third rail is fed into the motors and back again to the track return, and the auxiliary control system through which, by means of any one of the master controllers placed at each end of every motor car, the motorman actuates the switches which make and break connection in the main motor control system.

The interior of each car is lighted by twenty-six 16-c. p. incandescent lamps. Two 16-c. p. lamps are placed in each vestibule to light the platform and the steps. One 16-c. p. lamp is located in each marker, and a 50-c. p. lamp is placed in the headlight at each end of the car, which is fixed on the vestibule roof between the markers. The incandescent headlight is in series with a resistance and independent of all other circuits.

Heat is supplied by 24 electric heaters in the body of the car, and at each end there is one in the motorman's cab. Those in the body of the car are placed under the seats. Variation in the amount of heat is effected by means of two sets of heater coils, one of twice the capacity of the other. Either set or both sets may be cut in by switches, thus providing three degrees of heat.

All wires and cables are run in iron conduit. The sizes of wire used in the car wiring were No. 12 and No. 14 for the auxiliary control, and No. 0, No. 00, 144,000 c. m., and 250,000 c. m. for the main motor control circuit.

CONCLUSION.

The substations were first supplied with high-tension current from the overhead lines, and tested out, on April 27, 1905. The portable substations were first operated on June 12, 1905. Regular operation of the permanent substations began July 26, 1905. The steel cars have now been in operation for over a year.

The equipment of the steel cars and the design and construction of the substation system were carried out by Westinghouse, Church, Kerr & Co. The entire work was under the direction of Mr. George Gibbs, Chief Engineer of Electric Traction of the Long Island Railroad, subject to the approval of an electrical committee consisting of the chief operating officials of the road, with the president as chairman.

NOTES ON STREET PAVING.

In the course of a recent inquiry as to street traffic and wear of paving in a number of cities, some incidental information was obtained in regard to the use of different kinds of paving material. The relation of traffic to wear of street paving is discussed in our editorial columns, and a paper on censuses of street traffic in New York City is abstracted elsewhere in this issue. Our own reports from cities of different sizes are given briefly below:

ALTOONA, PA.—(Harvey Linton, City Engineer.) Asphalt block on concrete foundation, 17 years, has worn into holes in center of street. Sheet asphalt laid in 1889 has had some repairs; that laid in 1890, considerable repairs; Alcatraz, laid in 1900, badly worn, repairs now being made by street forces. Brick, "John Porter," laid in 1890, still in good condition, wear slight, light traffic. Brick is the popular pavement at this time. None other has been considered since 1900 by the petitioners for new paving.

AUGUSTA, GA.—(Nisbet Wingfield, Commissioner of Public Works.) We have never made an accurate account of the traffic, but as a general proposition we use Belgian block for the streets leading to the warehouses and stations, brick on the business streets, and macadam and gravel on the residence streets.

BALTIMORE, MD.—(B. T. Fendall, City Engineer.) There has never been any sort of record kept of the amount of traffic passing over any given street, nor have any measurements been made showing the wear incident to that traffic. As to the preference for the various classes of pavements, in the business section where very heavy trucks are used, and where the grades vary from 1 to 7%, about the only pavement that is considered is good granite block laid on a concrete base. The first cost of this pavement would be, under existing conditions, about $2.80 to $3 per sq. yd. Such a pavement, if not disturbed by plumber's cuts, would last 40 or 50 years. Of course, it would be very noisy. For a second-class of pavement, on residential streets, vitrified brick has been recommended for the light traffic. Where the traffic is somewhat heavier, and the improvements are of high grade, sheet asphalt is recommended unless the grades are over 3%. Where the grades are above 3% I would prefer bitulithic, and from what I have seen of this I am inclined to favor it before sheet asphalt, but our people consider it to be an experiment, and they do not appear to be willing to spend much money for this class of pavement. The fact of its being patented, and, therefore, a monopoly, also operates against it.

BURLINGTON, IA.—(Wm. Steyh, City Engineer.) There has been no record kept of traffic. Brick pavement is considered the best adapted for all purposes, and pavement laid in 1887 on one of the streets carrying the heaviest traffic is still in good condition.

CLEVELAND, O.—(William J. Carter, Chief Engineer of Department of Public Service.) We have no record of the amount of traffic over pavements, nor any record of the depth of wear. We prefer D. B. Medina sandstone in our business streets, and brick for the resident portion.

NEW YORK, N. Y., Borough of Brooklyn.—(Geo. W. Tillson, Chief Engineer of Bureau of Highways.) Very few records have been kept in this country of traffic of any pavements and those were nearly all made by the Barber Asphalt Paving Co. some years ago. In this borough, for heavy traffic granite blocks are generally used; for residence streets property owners almost invariably ask for asphalt; not so much on account of its cost as that this pavement is most suitable for resident streets.

NEW YORK, N. Y., Borough of Richmond.—Louis L. Tribus, Commissioner of Public Works.) The preference for paving on (1) business streets, (2) second-class streets, and (3) residence streets, is as follows: A, Cost; asphalt blocks for all three classes of streets; B, Durability; granite for class 1; asphalt blocks for the others; C, Noiselessness; asphalt blocks for all three classes; for Class 3 also macadam (frequently resurfaced), wooden blocks, and sheet asphalt. The use of the various kinds of paving are to be governed by the grades in each case.

OMAHA, NEB.—(Andrew Rosewater, City Engineer.) I know of no statistics whatever that would justify conclusions in this country as to the wear or length of life of different kinds of paving under a certain amount of traffic. This subject was gone into quite at length in London during long periods and the result of such tests is contained in an English publication known as "Roads and Streets" by Henry Law and D. K. Clark.

In reference to the kind of pavement preferred for residence and business streets in our city, up to a recent period sheet asphalt pavements have been preferred for both business and residence streets. Within the past few years, brick block has come into considerable use and stone pavements on concrete are only selected where the traffic is extremely heavy and the width of roadway is limited.

SOUTH BEND, IND.—(A. J. Hammond, City Engineer.) I have had to take up a part of one street laid in 1890 with South Bend paving brick made from a blue clay, burned white (brick 2 × 2¾ × 7¾ ins.) These brick, subjected to ordinary traffic of loads up to 5 tons, had worn in the middle of the street to a depth of about 1¼ to 1½ ins. and formed a flat arc over the original crown. The brick had worn uniformly so that the surface to the eye appeared a correct curve. These brick were taken up at an intersection in 1903 and the rest of the street is in service. The brick pavement has been a favorite here for all purposes largely because of the cost and durability. Our brick streets have for several years averaged less than $1.25 per sq. yd. There is a revival of interest in asphalt now because of the lower cost.

STEEL WHARVES WITH CONCRETE PIERS will be built at Manila by the United States Government. There will be two of these; one 650 ft. long and 110 ft. wide, and the other 600 ft. long and 70 ft. wide. Transverse plate girders will rest on the piers and carry between their webs longitudinal lines of 18-in. rolled steel joists for the floor framing. The floor will be of reinforced concrete in the form of a slab 4½ ins. thick, with expanded metal or wire netting embedded in it. Wooden fender piles will protect the face of the wharf and be connected by waling pieces. Spiral springs will also be inserted between the piles and the ends of the plate girders. For each pier between five and seven wooden piles, 65 to 75 ft. long, are driven so as to form a pyramid above the bottom, with the middle pile 15 or 20 ft. above the ground. Steel cylinders are then placed over the groups of piles and sunk 6 or 8 ft. into the bottom. Above the level of the top of the highest pile the cylinder tapers to a smaller top diameter. After the mud is pumped out vertical steel reinforcing bars are set in place, and the cylinders filled with concrete. The piers have a diameter of 6 ft., but taper to 3 ft. at the top. A concrete mixture of 1 part Portland cement, 2 parts sand, and 5 parts stone is used for them. Nearly 300 piers will be required as they are to be set 20 ft. apart transversely and 25 ft. apart longitudinally.

SOME EXPERIMENTS ON THE FRICTIONLESS ORIFICE.*

By Horace Judd and Roy S. King.†

These experiments, the results of which are herein recorded, had their beginning in the calibration of a 1-in. frictionless orifice.‡ During the early winter of 1905-06 the authors of this paper had occasion to standardize the orifice for use in the Experimental Engineering Laboratory of the Ohio State University. While the work of calibrating the orifice was in progress the thought occurred to the experimenters, that, if the area at the out the section, as it was believed to be, then the quantity discharged might be determined by taking the product of the velocity and the area of the jet.

Few data were at hand giving either the area at the least section or the position of the least section. The investigation of these two points alone was thought to be sufficient incentive to enlarge the scope of the experiments so as to include orifices of as large diameter as could be used with the available equipment for testing. We had hoped to include 4 and 5-in. orifices but were obliged to stop at the 2.5-in. orifice, although some work was done on the 3-in. orifice. The results on the of discharge for the verification of constants usually quoted; and it afforded a means of getting the coefficient of velocity for comparison with that obtained with the pitot tube. This would also show how near unity the constant of the pitot tube would approach. In the present experiments five orifices were used, of .75, 1, 1.5, 2 and 2.5 ins. nominal diameters. Each orifice was made by boring out the proper size hole in a cast-iron plate, ¼-in. thick, of circular form. The outer edge of the orifice was cut away at an angle of about 45° so that the edge presented to the water was $^1/_{32}$-in. thick. Each orifice plate could be slipped into place and securely clamped in the

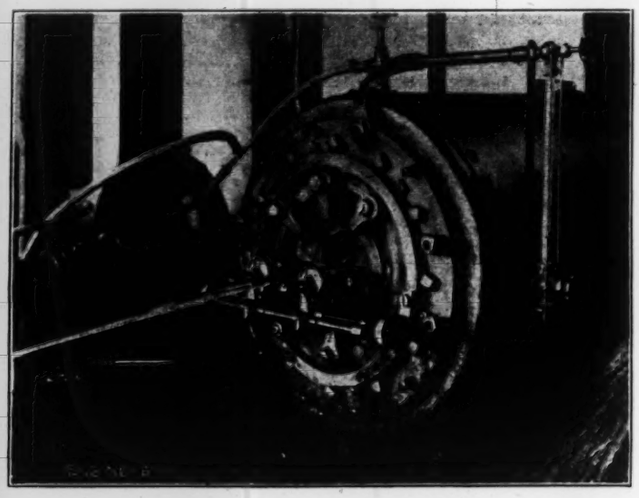


FIG. 1. END OF DRUM, WITH HYDRAULIC CALIPERS IN HORIZONTAL POSITION, AND WITH STREAM FLOWING FROM ¾-IN. ORIFICE.

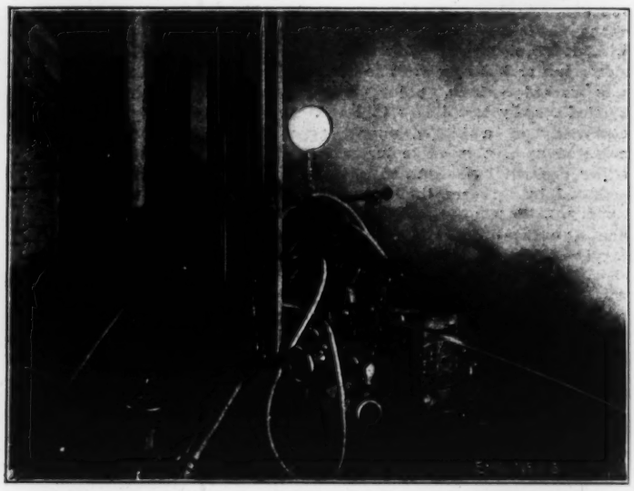


FIG. 2. STAND-PIPE AND DRUM; MERCURY COLUMNS (A AND B); PITOT TUBE (D).

FIGS. 1 AND 2. VIEWS OF APPARATUS USED IN EXPERIMENTS ON THE FRICTIONLESS ORIFICE.

least section were known the orifice might be made to serve as a convenient and accurate form of water meter by taking the velocity at the least section by means of a pitot tube. If the velocity should be uniform through-out the section, as it was believed to be, then the quantity discharged might be determined by taking the product of the velocity and the area of the jet.

Few data were at hand giving either the area at the least section or the position of the least section. The investigation of these two points alone was thought to be sufficient incentive to enlarge the scope of the experiments so as to include orifices of as large diameter as could be used with the available equipment for testing. We had hoped to include 4 and 5-in. orifices but were obliged to stop at the 2.5-in. orifice, although some work was done on the 3-in. orifice. The results on the 3-in. orifice were found to be somewhat erratic and have not been included in this record. This irregularity we are inclined to believe was due to the increased velocity of the water in passing into and through the drum leading to the orifice; also the pumps at our command were running too near the limit of their capacity to admit of steady control.

The series of experiments on frictionless orifices was undertaken for the purpose of determining the following points: (1) The coefficient of discharge by calibration. (2) The shape of the jet in the vicinity of the least section. (3) The diameter of the jet at the least section. (4) The effect of increase in static pressure on the diameter at the least section and the shape of the jet. (5) The velocity in the least section by use of the pitot tube.

COEFFICIENT OF DISCHARGE BY CALIBRATION.

The calibration of the orifices was necessary for the purpose of finding by direct measurement the coefficient of discharge for the verification of constants usually quoted; and it afforded a means of getting the coefficient of velocity for comparison with that obtained with the pitot tube. This would also show how near unity the constant of the pitot tube would approach. In the present experiments five orifices were used, of .75, 1, 1.5, 2 and 2.5 ins. nominal diameters. Each orifice was made by boring out the proper size hole in a cast-iron plate, ¼-in. thick, of circular form. The outer edge of the orifice was cut away at an angle of about 45° so that the edge presented to the water was $^1/_{32}$-in. thick. Each orifice plate could be slipped into place and securely clamped in the end of a 24-in. drum, forming a smooth surface on the inside. (See the views, Figs. 1 and 2, and the sections, Fig. 3.)

The desired pressure at the orifice was maintained by means of a 10, 16, 12 × 10-in. stroke duplex compound Worthington fire pump, capacity 1,000 gals. per min., and a 12, 7 × 12-in. stroke Knowles pump, capacity 150 gals. per min. The water as it came from the pumps was stored in a 24-in. vertical stand-pipe, 25 ft. high and of about 80 cu. ft. capacity. At the bottom of the stand-pipe a drum 24 ins. in diameter and 48 ins. long projected horizontally, to the end of which was fastened the orifice to be tested. The connection between the stand-pipe and drum was narrowed down to a diameter of 13 ins. This was known to be a detriment to the steady flow of water in the drum, especially at high velocities where it gave rise to much eddying and cross currents. This was very noticeable in the stream from the orifice (es-

*Slightly condensed from a paper read before the American Association for the Advancement of Science, at Ithaca, N. Y., July, 1906.

†Assistant Professor of Experimental Engineering, Ohio State University, Columbus, Ohio.

‡The term "frictional orifice" is one which has been commonly used in the mechanical laboratory of the Ohio State University for many years. It is the same as what the text-books call "an orifice in a thin plate," and indicates that there is no appreciable loss by friction by a stream of water flowing through such an orifice. So far as we know, the term was first employed, as early as 1873, by Prof. S. W. Robinson, Emeritus Professor of Mechanical Engineering at Ohio State University, in his report on "Measurement of Gas Wells and Other Gas Streams, and the Piping of Natural Gas," being Chapter IX. of Vol. VI. of the Geological Reports of Ohio, and will be found on page 552.

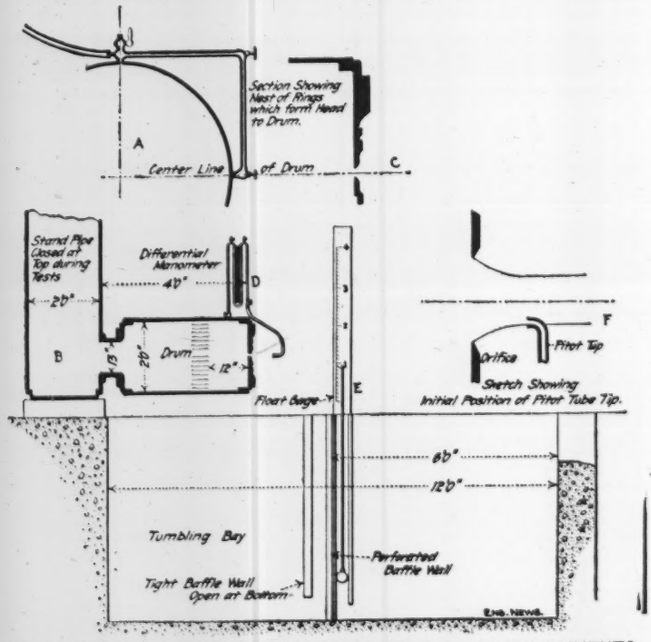


FIG. 3. GENERAL ARRANGEMENT OF APPARATUS FOR EXPERIMENTS ON THE FRICTIONLESS ORIFICE.

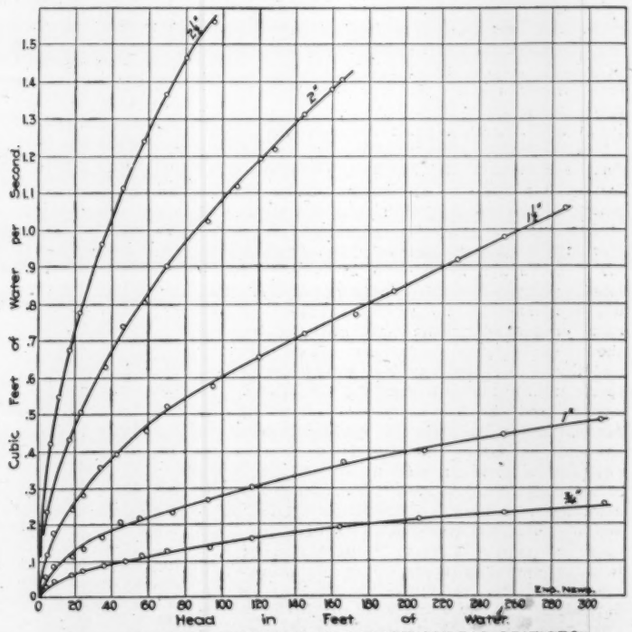


FIG. 4. DISCHARGE CURVES FOR FRICTIONLESS ORIFICES.

TABLE I.—MAXIMUM, MINIMUM AND AVERAGE READINGS OF STATIC PRESSURES TAKEN DURING CALIBRATION OF ORIFICES.

0.751-in. Orifice.												
Maximum	7.8	10.7	15.8	20.3	25.6	30.8	40.6	50.3	71.4	90.8	112.0	134.2
Minimum	7.2	9.8	14.8	19.8	24.4	29.5	39.8	49.8	69.9	88.0	111.0	131.6
Av. 8 readings	7.5	10.3	15.2	20.1	25.0	30.1	40.1	50.3	70.3	89.6	111.5	133.2
1.025-in. Orifice.												
Maximum	7.8	10.6	15.3	20.8	25.4	30.4	40.5	51.5	70.8	91.9	111.6	133.5
Minimum	6.9	9.8	14.5	19.8	24.5	29.8	39.6	49.4	69.8	89.6	110.3	130.5
Av. 8 readings	7.4	10.2	14.9	20.3	24.8	30.1	39.9	49.9	70.2	90.6	111.0	131.2
1.507-in. Orifice.											**Av. 12 readings.**	
Maximum	7.3	10.1	15.5	25.3	31.0	42.9	52.0	73.0	84.3	62.5	111.3	....
Minimum	6.5	9.5	14.4	25.2	30.0	40.1	51.5	69.5	82.6	62.0	110.9	....
Av. 6 readings	6.9	9.8	15.1	25.2	30.2	41.1	51.9	71.3	83.6	62.1	111.1	....
1.9975-in. Orifice.												
Maximum	8.0	10.5	15.6	20.8	25.3	30.3	42.0	58.5	62.4	72.0	75.0	....
Minimum	7.3	9.0	15.4	20.2	24.0	30.0	38.0	52.3	61.3	61.0	60.5	....
Av. 12-15 readings	7.5	9.8	15.5	20.4	24.9	30.2	39.9	55.8	62.2	68.6	69.6	....
2.500-in. Orifice.												
Maximum	3.0	5.1	7.8	10.0	15.1	20.0	25.1	30.1	35.2	41.2	....	....
Minimum	3.0	4.9	7.5	9.5	14.9	20.0	24.9	29.1	34.8	40.8	....	....
Av. 11 readings	3.0	5.0	7.6	9.9	15.0	20.0	25.0	30.0	35.0	41.0	....	....

TABLE II.—CALIBRATION OF ORIFICES.

.751-in. Orifice.									
Number of run	1	2	3	4	5	6	7	8	9
Length of run, in secs	1,673	1,114	1,283	1,380	1,125	992	897	1,217	1,070
Temperature, F°	59	59	59	60	60	59	59	56	57
True gage pressure, lbs	1.78	3.20	7.26	10.22	15.30	19.05	23.55	28.95	36.80
Head, in ft	4.10	8.99	16.75	23.60	35.33	43.85	54.25	66.80	84.70
Cu. ft. water, total	50.3	50.2	79.1	101.0	101.0	99.2	99.8	149.8	149.3
Cu. ft. water, per sec	.03006	.04506	.06165	.0732	.0898	.1000	.1112	.1231	.1397
Least diam. jet (av.), ins	.5882	....	....	....	....	....	....	....	....
Least area jet (av.), sq. ins	.2715	....	....	....	....	....	....	....	....
Position least diam	8 in.	from plane of orifice.							
Velocity, ft. per sec., at least sect'n by calibration	15.87	23.88	32.70	38.83	47.63	53.04	59.98	65.29	74.10
Velocity, ft. per sec., at least section, theoretical ($V = \sqrt{2gh}$)	16.06	23.98	32.84	38.98	47.70	53.20	59.08	65.55	73.84
Coefficient of velocity by calibration	.992	.994	.995	.995	.998	.996	.998	.995	1.003
Coefficient of velocity by Pitot tube	.9996	.9998	.99997	.99997	.99994	.99999	.99995	.99995	.99996
Coefficient of contraction	.6134	....	....	....	....	....	....	....	....
Coefficient of discharge by calibration	.6085	.6102	.6103	.6105	.6120	.6111	.6119	.6104	.6150
Coefficient of discharge by Pitot tube	.6134	....	....	....	....	....	....	....	....
1.025-in. Orifice.									
Number of run	1	2	3	4	5	6	7	8	9
Length of run, in secs	1,780	1,190	1,284	1,090	895	740	932	849	943
Temperature, F°	59	59	58	58	59	58	58	58	59
True gage pressure, lbs	1.74	3.85	7.52	10.45	15.55	22.45	25.30	30.70	39.96
Head, in ft	4.02	8.87	17.35	24.65	35.80	51.85	58.20	70.75	92.45
Cu. ft. water, total	100.9	99.0	149.8	150.7	150.7	150.0	199.9	199.8	249.7
Cut. ft. water, per sec	.0567	.0832	.1166	.1382	.1684	.2026	.2145	.2353	.2647
Least diam. jet (av.), ins	.8015	....	....	....	....	....	....	....	....
Least area jet (av.), sq. ins	.5045	....	....	....	....	....	....	....	....
Position least diam	8 in.	from plane of orifice.							
Velocity, ft. per sec., at least sect'n by calibration	16.11	23.74	33.28	39.45	47.98	57.80	61.14	67.10	76.25
Velocity, ft. per sec., at least section, theoretical ($V = \sqrt{2gh}$)	16.08	23.90	33.42	39.82	48.00	57.80	61.22	67.50	77.10
Coefficient of velocity by calibration	1.001	.994	.996	.991	1.0005	1.0005	.999	.995	.997
Coefficient of velocity by Pitot tube	.99996	.99988	.9998	.9999	.99993	.99999	.99994	.99996	.99998
Coefficient of contraction	.6115	....	....	....	....	....	....	....	....
Coefficient of discharge by calibration	.6122	.6078	.6092	.6059	.6117	.6118	.6108	.6084	.6096
Coefficient of discharge by Pitot tube	.6115	....	....	....	....	....	....	....	....
1.507-in. Orifice.									
Number of run	1	2	3	4	5	6	7	8	9
Length of run, in secs	2,169	855	1,031	884	704	636	547	486	435
Temperature, F°	61	60	58	61	57	60	58	57	57
True gage pressure, lbs	1.76	3.73	7.16	9.66	15.19	18.88	25.20	30.56	39.45
Head, in ft	4.07	8.62	16.53	22.25	35.66	43.61	58.20	70.56	91.10
Cu. ft. water, total	260.4	150.3	250.0	250.2	249.8	249.8	249.6	249.7	249.6
Cut. ft. water, per sec	.1201	.1755	.2425	.2830	.3548	.3930	.4563	.5138	.5735
Least diam. jet (av.), ins	1.1722	....	....	....	....	....	....	....	....
Least area jet (av.), sq. ins	1.0797	....	....	....	....	....	....	....	....
Position least diam	1.2 ins.	from plane of orifice.							
Velocity, ft. per sec., at least sect'n by calibration	16.02	23.41	32.34	37.75	47.32	52.41	60.84	68.55	76.50
Velocity, ft. per sec., at least section, theoretical ($V = \sqrt{2gh}$)	16.19	23.55	32.62	37.86	47.92	53.01	61.25	67.42	76.60
Coefficient of velocity by calibration	1.005	1.005	1.006	1.006	1.006	1.006	1.006	1.021	1.005
Coefficient of velocity by Pitot tube	.99988	.99988	.99994	.99996	.99993	.99997	.99993	.99993	.99999
Coefficient of contraction	.6051	....	....	....	....	....	....	....	....
Coefficient of discharge by calibration	.6084	.6084	.6087	.6086	.6086	.6086	.6086	.6180	.6084
Coefficient of discharge by Pitot tube	.6051	....	....	....	....	....	....	....	....
1.9975-in. Orifice.									
Number of run	1	2	3	4	5	6	7	8	9
Length of run, in secs	1,255	934	651	592	395	340	310	276	194
Temperature, F°	59	56	56	56	57	57	57	57	57
True gage pressure, lbs	2.16	3.93	7.70	10.06	15.63	20.37	24.95	30.29	39.83
Head, in ft	5.00	9.08	17.79	23.24	36.12	47.02	57.70	69.99	92.01
Cu. ft. water, total	300.3	300.0	310.1	300.0	249.7	249.7	249.4	249.4	199.2
Cu. ft. water, per sec	.2390	.3212	.4476	.5070	.6320	.7344	.8045	.904	1.028
Least diam. jet (av.), ins	1.5581	....	....	....	....	....	....	....	....
Least area jet (av.), sq. ins	1.9055	....	....	....	....	....	....	....	....
Position least diam	1.5 ins.	from plane of orifice.							
Velocity, ft. per sec., at least sect'n by calibration	18.06	24.34	33.50	38.32	47.75	55.46	60.76	68.56	77.7
Velocity, ft. per sec., at least section, theoretical ($V = \sqrt{2gh}$)	17.94	24.18	33.86	38.68	48.20	55.03	60.96	67.15	77.00
Coefficient of velocity by calibration	1.0066	1.0065	1.004	1.0065	1.0004	1.001	1.0002	1.0001	1.0001
Coefficient of velocity by Pitot tube	.9998	.99988	.99988	.99996	.99998	.99997	.99997	.99998	.99998
Coefficient of contraction	.6082	....	....	....	....	....	....	....	....
Coefficient of discharge by calibration	.6084	.6083	.6090	.6083	.6082	.6088	.6081	.6080	.6080
Coefficient of discharge by Pitot tube	.6082	....	....	....	....	....	....	....	....
2.500-in. Orifice.									
Number of run	1	2	3	4	5	6	7	8	9
Length of run, in secs	600	480	360	360	300	210	210	180	150
Temperature, F°	64	64	64	64	64	64	64	64	64
True gage pressure, lbs	2.93	4.95	7.61	10.03	15.05	20.03	25.03	30.05	40.25
Head, in ft	6.78	11.54	17.59	23.17	34.77	46.27	57.81	69.42	93.00
Cu. ft. water, total	254.8	264.5	244.2	280.6	288.4	233.5	260.7	246.1	235.2
Cu. ft. water, per sec	.4247	.5511	.6780	.7792	.9613	1.112	1.242	1.368	1.565
Least diam. jet (av.), ins	1.9295	....	....	....	....	....	....	....	....
Least area jet (av.), sq. ins	2.9245	....	....	....	....	....	....	....	....
Position least diam	1.8 ins.	from plane of orifice.							
Velocity, ft. per sec., at least sect'n by calibration	20.90	27.10	33.38	38.35	47.35	54.72	61.16	67.35	77.10
Velocity, ft. per sec., at least section, theoretical ($V = \sqrt{2gh}$)	20.89	27.28	33.66	37.91	47.35	54.55	61.00	66.86	77.35
Coefficient of velocity by calibration	1.0008	1.000	1.0005	1.0008	.9998	1.0008	1.0000	1.0000	.995
Coefficient of velocity by Pitot tube	.9999	.9998	.99988	.99996	.99995	.99998	.99999	.99998	.99999
Coefficient of contraction	.5955	....	....	....	....	....	....	....	....
Coefficient of discharge by calibration	.5960	.5956	.5958	.5960	.5950	.5960	.5955	.5955	.5952
Coefficient of discharge by Pitot tube	.5955	....	....	....	....	....	....	....	....

pecially from the larger orifices) by the appearance of spiral currents moving around the jet and also by the swaying of the jet from side to side. We remedied this to a great extent so that the spiral jets ceased and the stream was much more quiet and swayed but little, by placing a curtain of thin metal pipes 12 ins. in the rear of the orifice plate. This curtain was made up of thin zinc cylinders 8 ins. long and about 2.5 ins diameter, held in place sufficiently secure by simply wedging them in together as tightly as possible.

The water discharged by the orifice was measured directly in a calibrated cistern, or bay, of about 100 sq. ft. area and 3,000 gals. capacity. The rise in the bay was indicated by a float gage whose scale was graduated in .01 ft. For the 1.5-in. orifice, and also those above, where the cistern would nearly fill before conditions became constant, we made use of deflector pipes so that the jet could be shifted at will into either of the two bays without sensible loss of water.

Pressure at the orifice for 7.5 lbs. and upwards was read from a Crosby standard test gage, 6-in. face, of the Bourdon spring type, graduated in pounds, which was attached to the upper side of the drum. The pipe to which the gage was attached was also in communication with the side of the drum at a point on the level with the center of the orifice. (See Figs. 1 and 3.) A glass tube formed part of this line of communication, so that the water in the drum could be kept at any desired height by means of an air cock near the gage connection. In this way it is known definitely just to what height the water rose in the gage connection, so that an accurate correction could be made in the pressure readings for the height of the gage above the center of the orifice. During the whole series of the trials nearly all of the air was relieved from the top of the drum, there being at no time more than 1-in. height of air remaining at the top of the horizontal drum, as shown by the gage glass column. The other static heads below 7.5 lbs. were read directly from a water column placed at the side of the drum.

An attempt was made at all times to keep the work as free from errors as the character and condition of the apparatus would allow. The pressure gage could be read within 0.1 lb. For the higher pressures this would make no sensible error; and for pressures of 10 lbs. and under the error would not exceed 1%.

The bay in which the water was discharged had been carefully calibrated by filling with a known weight of water at a known temperature. The float scale could be read with accuracy to within 0.005 ft. This would mean no greater error than 0.01 ft. And since 1 ft. rise is equivalent to very nearly 100 cu. ft. volume the greatest error is not likely to exceed 1 cu. ft. per 100. The quantities measured range from 50 to 300 cu. ft. The smallest quantities were discharged from the smallest sized orifices and required the greatest lengths of time; or about $\frac{1}{6}$ the quantity in 10 times the time as for the largest orifice, which would bring the error well within 1% for those runs of less than 100 cu. ft. and for the other runs within a fraction of 1%.

The time was taken with a stop watch which was checked with a watch known to be correct for the greatest range of time required for a run. The error in starting and stopping is thought to be within $\frac{1}{5}$ sec. This will appreciably affect only those runs whose time was less than 200 secs. For the shortest length of time, 150 secs., the error would not be over 0.3%.

The pressure was regulated by means of an outlet pipe leading from the stand-pipe into the suction bay. Readings on the pressure gage were taken every 0.2-ft. rise on the float gage for most of the runs, excepting for the $\frac{3}{4}$ and 1-in. orifices, where pressure readings were taken at equal intervals of time giving at least ten readings per run.

Computations for the most part were made with a four-place logarithm table, with the exception of the calculations for the coefficient of velocity by the pitot tube, where a five-place table was used to locate the last figure in the result. Later the results were checked by a five-place slide rule. Table I. shows the maximum, minimum and average static pressure readings. In Table II. will be seen the average data and the results for the coefficients of the orifices found both by calibration and also by means of the pitot tube.

The coefficient of discharge was computed by the usual formula,

$$q = CA\sqrt{2gH},$$

where q = quantity discharged by actual measurement,
C = coefficient of discharge,
A = area of orifice in sq. ft., and

$\sqrt{2gH}$ = theoretical velocity equivalent to the static head H.

Curves have been plotted showing the quantity discharged in cu. ft. per second for different heads in feet of water. (See Fig. 4.)

Our results do not show any great change in the coefficient of discharge for increase in head, while authorities give a decrease in the coefficient for increase in head.[a]

[a] Merriman's "Hydraulics," 1903, p. 551; also Bovey's "Hydraulics," 1904, p. 40.

Merriman gives values from determinations by Hamilton Smith which show for a ½-in. orifice a decrease in the coefficient of 2½% and for a 2½-in. orifice a decrease of about 1%, for heads ranging from 5 to 100 ft. Bovey finds about 1% drop in the coefficient for a ½-in. orifice, 5 to 20-ft. head, which would not be over 1½% drop for 5 to 100-ft. head.

Our coefficients of discharge are as follows:

Orifice, ins.....	¾	1	1.5	2	2½
Coef. Disch.....0.6111	0.6097	0.6085	0.6083	0.5956	
In previous experiments[b] for 5 ft.	0.6119	0.6112			

Bazin[c] on a 7.87-in. orifice and 3.28-ft. head obtained 0.598.

Merriman gives for diam., ins.....	0.84	1.2	2.4
Average coef. discharge for heads from 5-100 ft.....	0.597	0.596	0.595

Bovey[d] found for:
½-in. diameter, an av. coef. disch. of 0.6053 for 5 to 20 ft. heads.
1-in. diameter, an av. coef. disch. of 0.6000 for 5 to 20 ft. heads.

It is interesting to note that Bovey eliminated any error due to time readings by using an electric chronograph. Our results seem to indicate a decrease in the coefficient of discharge of about 2½% with increase in size of orifice from ¾ to 2½ ins. This agrees fairly well with values given by Merriman and Bovey. Our average value (0.6096) for the coefficients of discharge runs about 1.7% higher than that (0.596) given by Merriman for the same range of head and same size of orifices. As compared with that (0.6000) given by Bovey for the 1-in. orifice our value (0.6097) is 1.6% high.

SHAPE OF THE JET IN THE VICINITY OF THE LEAST SECTION.

In order to locate the position of the contracted vein, or least section, we had to examine the contour of the jet until a minimum section was found.

The shape of the jet and also the average diameter at the least section was obtained by the use of a special micrometer calipers[e] which we will speak of as "hydraulic calipers." (See Fig. 1.) These calipers were designed by Prof. Wm. T. Magruder and are attached to the head of the drum to which the orifice is fastened and can be rotated at will in the plane perpendicular to the jet. A heavy brass ring is fitted into a groove in the head of the drum concentric with the orifice. This ring is held rigid in place by six brass lugs. A pair of carriages is moved on parallel rods by micrometer screws which move in a plane normal to the jet. Each carriage supports a micrometer screw with a brass point attached which can be moved in a direction parallel to the jet. The brass points are bent in such a manner as to present the points as nearly normal to the stream as possible. With our points we were able to take measurements for a distance of 2 ins. along the jet beginning $\frac{1}{10}$-in. from the plane of the orifice and at any point in any plane normal to the jet within these limits.

The micrometer screws in the calipers were checked by means of a B. & S. micrometer gage and found to be accurate within 0.001-in. These carriages were also checked for parallelism of feed to and from the plane of the orifice, and where necessary, corrections were made.

Diameters of the jet were taken for at least ten points along the contour until a diameter's distance from the plane of the orifice was reached for all orifices except the 2.5-in. where the farthest distance out was 2 ins.

It was thought that the position of the least section could be located with sufficient accuracy by plotting the horizontal contours only: later while verifying some of the horizontal contours, vertical readings were also taken and on plotting them we obtained practically the same results as those obtained from the horizontal measurements. Table III. is given as a sample set of contour diameter readings. They are purposely chosen from those taken on the 2.5-in. orifice to show the range of variation with the jet issuing from the orifice with its greatest fluctuations. It will be noticed that each net diameter is the result of ten independent observations. In most cases ten observations were taken for each net diameter, except for the check runs made later, when only six readings per diameter were taken.

For the smaller diameters where the stream was more quiet we believe our error to be no greater than 0.003-in. and for the largest orifice not to exceed 0.01-in.; which would mean an error within 0.8 of 1%.

Fig. 5 shows the actual shape of the jets, enlarged four times, plotted from values given in Table IV. Values for the 7.5-lb. static pressures are plotted below and those for the 40-lb. static pressures above the center line. The contour curves are also plotted relatively; that is, both the distance from the plane of the orifice and the observed radius of the jet are given in terms of the radius of the orifice and are plotted accordingly. Table V. gives the data of plotting the relative contour curves shown on Fig. 6. The values given in the table include all diameters found from both the horizontal and vertical observations. The curves are arranged (Fig. 6) one above the other in the order of the sizes of the orifices and are placed distances apart proportional also to the sizes. Thus for the ¾-in. contour curve the distance below the 1-in. contour curve is one-half that between the other curves. For some distance after the least section has apparently been reached the diameter of the jet remained practically constant. An average horizontal line has been drawn through such points in the outer portion of each curve. The point where the contour curve departed from the horizontal line was taken as the position of the least section. A smooth curve has been traced through the points thus located on each curve, which will serve to indicate the effect of the diameter of the orifice on the position of the least section. From an inspection of the latter curve it will be seen that the stream from the smallest orifice does not reach its least

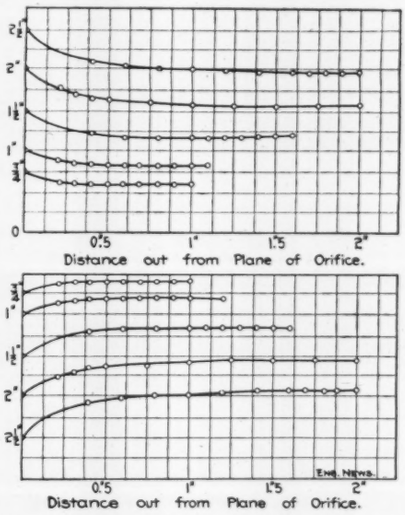


Fig. 5. Shapes of Jets From Orifices of Different Sizes.

[b] Boyd and Judd, Eng. News, March 31, 1904.
[c] Bazin, Contraction of the Liquid Vein, J. Wiley & Sons, 1896, p. 9.
[d] Bovey's "Hydraulics," page 40.
[e] "An Hydraulic Micrometer Caliper," Wm. T. Magruder, "Science," March 4, 1904, page 364.

TABLE III.—Observed Data for Contour or Shape of Orifice, for 2.5-in. Orifice; Readings Taken on Horizontal Diameter, 40-lbs. Head.

Distance out from plane.	Micrometer read'gs. Right.	Left.
.4-in.	1.221	1.305
	1.223	1.307
	1.226	1.308
	1.224	1.307
	1.228	1.306
	1.2240	1.3068
	.1670	.2888
	1.0574	1.0178
	Net diam. =	2.0242
.6-in.	1.201	1.279
	1.201	1.282
	1.200	1.283
	1.198	1.278
	1.198	1.280
	1.1996	1.2804
	.1670	.2888
	1.0326	.9916
	Net diam. =	2.0242
.8-in.	1.187	1.263
	1.186	1.263
	1.191	1.262
	1.184	1.261
	1.183	1.262
	1.1862	1.2622
	.1670	.2888
	1.0192	.9734
	Net diam. =	1.9926
1.0-in.	1.177	1.247
	1.174	1.253
	1.177	1.252
	1.176	1.251
	1.175	1.249
	1.1758	1.2504
	.1670	.2888
	1.0088	.9616
	Net diam. =	1.9704
1.2-in.	1.170	1.242
	1.169	1.247
	1.173	1.251
	1.171	1.245
	1.172	1.244
	1.1710	1.2458
	.1670	.2888
	.9570	.9570
	Net diam. =	1.9610
1.4-in.	1.168	1.240
	1.164	1.240
	1.166	1.241
	1.163	1.239
	1.162	1.238
	1.1646	1.2396
0 =	0.1670	.2888
	.9976	.9508
	Net diam. =	1.9484
1.6-in.	1.164	1.233
	1.160	1.234
	1.162	1.234
	1.162	1.232
	1.162	1.229
	1.1620	1.2324
	.1670	.2888
	.9950	.9436
	Net diam. =	1.9386
1.7-in.	1.164	1.230
	1.161	1.230
	1.163	1.228
	1.162	1.228
	1.161	1.227
	1.1622	1.2286
	.1670	.2888
	.9952	.9398
	Net diam. =	1.9350
1.8-in.	1.160	1.234
	1.158	1.229
	1.157	1.236
	1.160	1.236
	1.162	1.238
	1.1594	1.2346
	.1670	.2888
	.9924	.9458
	Net diam. =	1.9382
1.9-in.	1.165	1.231
	1.167	1.232
	1.160	1.236
	1.168	1.238
	1.163	1.237
	1.1646	1.2348
	.1670	.2888
	.9976	.9460
	Net diam. =	1.9436

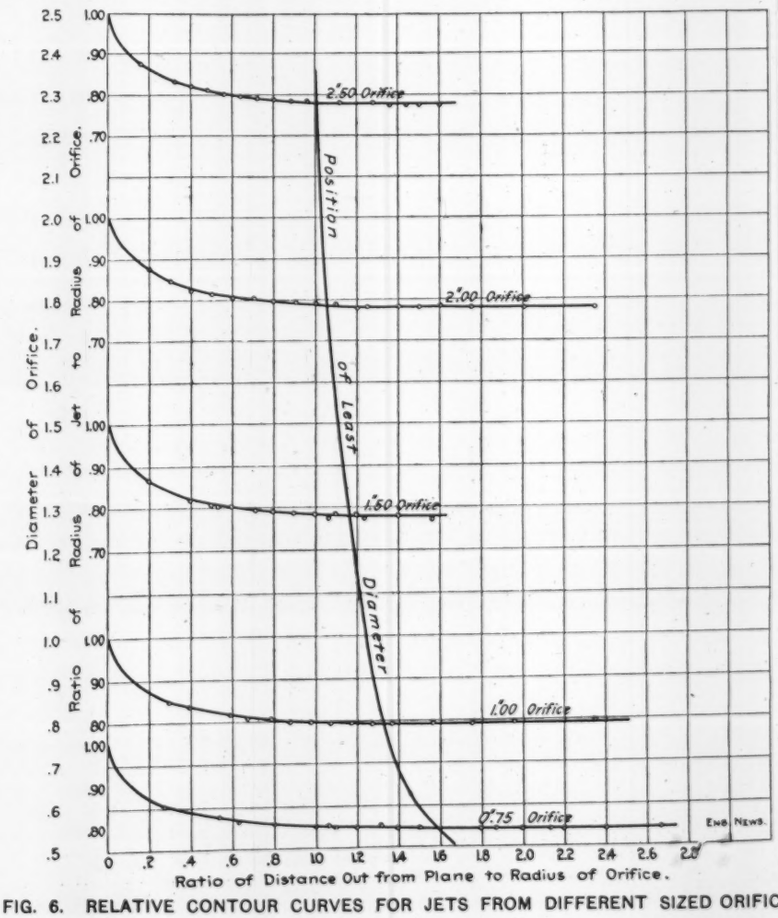


FIG. 6. RELATIVE CONTOUR CURVES FOR JETS FROM DIFFERENT SIZED ORIFICES.

TABLE IV.—DIAMETERS OF ORIFICES FOR VARIOUS DISTANCES FROM PLANES.

.751-in. Orifice.

Distance out from plane.....	.2	.3	.4	.5	.6	.7	.8	.9	1.0		
7.5 lbs. head.....	.6207	.6116	.6048	.6017	.5989	.5969	.5941	.5943	.5958	....	....
40 lbs. head.....	.6220	.6049	.6000	.5973	.5942	.5923	.5911	.5915	.5951	....	....

1.025-in. Orifice.

Distance out from plane.....	.2	.3	.4	.5	.6	.7	.8	.9	1.0	1.2	
7.5 lbs. head.....	.8703	.8479	.8267	.8200	.8177	.8146	.8123	.8117	.8178	.8200	....
40 lbs. head.....	.8799	.8428	.8302	.8211	.8185	.8145	.8116	.8096	.8098	.8106	....

1.507-in. Orifice.

Distance out from plane.....	.4	.6	.8	1.0	1.1	1.2	1.3	1.4	1.5	1.6	
7.5 lbs. head.....	1.2046	1.1817	1.1649	1.1615	1.1607	1.1557	1.1573	1.1587	1.1593	1.1543	....
40 lbs. head.....	1.2133	1.1829	1.1707	1.1693	1.1631	1.1597	1.1625	1.1637	1.1687	1.1699	....

1.9975-in. Orifice.

Distance out from plane.....	.20	.30	.40	.50	.75	1.00	1.25	1.50	1.75	2.00	2.25
7.5 lbs. head.....	1.7269	1.6933	1.6477	1.6286	1.5790	1.5688	1.5543	1.5508	1.5538	1.5562	1.5476
40 lbs. head.....	1.7419	1.6866	1.5612	1.6323	1.6021	1.5773	1.5733	1.5623	1.5623	1.5608	1.5593

2.500-in. Orifice.

Distance out from plane.....	.4	.6	.8	1.0	1.2	1.4	1.6	1.7	1.8	1.9	2.0
7.5 lbs. head.....	2.0894	2.0242	1.9926	1.9790	1.9662	1.9386	1.9364	1.9266	1.9266	1.9264	1.9236
40 lbs. head.....	2.0752	2.0242	1.9926	1.9704	1.9610	1.9484	1.9476	1.9330	1.9302	1.9418	1.9426

TABLE V.—DATA FOR PLOTTING RELATIVE CONTOUR CURVES; AVERAGE RESULTS FOR THE 7.5-LB. AND 40-LB. HEADS, HORIZONTAL AND VERTICAL.

r = .3755		r = .5125		r = .7535		r = .9988		r = 1.250	
Ratio dist. out from plane of orifice to radius. (A.)	Av. ratio of radius of jet to radius of orifice. (B.)	A.	B.	A.	B.	A.	B.	A.	B.
.267	.853	.293	.849	.199	.867	.200	.876	.160	.875
.533	.828	.391	.838	.398	.819	.301	.846	.320	.832
.626	.814	.585	.817	.498	.809	.400	.823	.400	.821
.799	.810	.683	.808	.531	.804	.501	.816	.480	.811
.800	.809	.781	.804	.597	.803	.590	.800	.560	.803
.999	.800	.878	.799	.710	.795	.701	.803	.640	.797
1.065	.802	.976	.797	.796	.791	.751	.796	.720	.791
1.092	.799	1.073	.794	.896	.788	.799	.798	.800	.789
1.198	.797	1.171	.794	.995	.784	.903	.794	.881	.789
1.318	.795	1.268	.792	1.062	.775	1.000	.791	.960	.785
1.332	.794	1.366	.792	1.095	.783	1.102	.789	1.120	.781
1.398	.793	1.561	.792	1.194	.782	1.202	.779	1.280	.780
1.598	.794	1.756	.792	1.237	.772	1.252	.783	1.360	.773
1.797	.791	1.952	.794	1.395	.780	1.402	.781	1.440	.773
1.867	.791	2.342	.796	1.460	.771	1.503	.780	1.520	.775
1.997	.790	....	...	1.593	.777	1.602	.786	1.600	.774
2.132	.789	....	...	1.725	.770	1.753	.780	....	...
2.192	.790	....	...	1.858	.771	2.003	.780	....	...
2.400	.791	....	...	2.037	.770	2.253	.778	....	...
2.665	.793	....	...	2.124	.771	....	...	....	...

section until a distance of 1.6 radii from the plane of the orifice, while for the 2.5-in. orifice the least section seems to lie in the vicinity of one radius out.

The distance out to the least section increases more rapidly as the size of the orifice decreases, so that for orifices below ¾-in. we would expect to find the least section a diameter out and perhaps more, while for orifices above 2.5 ins. we would look for the smallest section just within a radius distance from the plane of the orifice.

Most text-books on hydraulics give the distance out for the least section to be one radius. M. Bazin[f] in his investigation on the contraction of the liquid vein could find no "minimum section." He found that the vein continued to contract through very slightly, as the distance from the orifice increased. His experiments included transverse section measurements on a 7.87-in. orifice, at a low head of 3.28 ft., carried out for a distance of about 5.2 *radii* from the plane of the orifice.

[f] "Experiments on the Contractions of the Liquid Vein," Bazin-Trautwine.

TABLE VI.—Readings Taken from Observed Data for diameters at Least Section for 1-in. Orifice for 5 ft.-lbs., 40-lb. and 100-lb. Heads.

Micrometer Readings for 5-ft. Head.

Horizontal.		45° up.		Vertical.		45° down.	
Left.	Right.	Left.	Right.	Left.	Right.	Left.	Right.
.5970	.6683	.6025	.6625	.5870	.6740	.5815	.6735
.5974	.6657	.6030	.6620	.5834	.6755	.5850	.6742
.5984	.6645	.6025	.6595	.5835	.6724	.5830	.6755
.5980	.6674	.6010	.6600	.5873	.6730	.5828	.6737
.5965	.6680	.5990	.6585	.5845	.6720	.5834	.6746
.5975	.6668	.6016	.6605	.5852	.6734	.5833	.6743
.1690	.2920	.1690	.2920	.1690	.2920	.1690	.2920
.4285	.3748	.4326	.3685	.4162	.3814	.4143	.3823
Net diam. =	.8033		.8011		.7976		.7966
			40-lb. Head.				
.5948	.6780	.5975	.6620	.5960	.6730	.5900	.6740
.5998	.6770	.5970	.6630	.5940	.6730	.5889	.6750
.5994	.6665	.5990	.6644	.5960	.6735	.5889	.6735
.5965	.6678	.5985	.6660	.5922	.6725	.5889	.6750
.5994	.6670	.5995	.6605	.5958	.6725	.5888	.6740
.5980	.6713	.5983	.6642	.5948	.6729	.5891	.6747
.1690	.2920	.1690	.2920	.1690	.2920	.1690	.2920
.4290	.3793	.4293	.3722	.4258	.3809	.4201	.3827
Net diam. =	.8083		.8015		.8067		.8028
			100-lb. Head.				
.6005	.6645	.5980	.6683	.5910	.6730	.5950	.6724
.6005	.6664	.5930	.6637	.5900	.6720	.5880	.6720
.5995	.6670	.5964	.6605	.5890	.6740	.5889	.6715
.5997	.6664	.5955	.6625	.5915	.6720	.5896	.6720
.5997	.6675	.5960	.6627	.5915	.6725	.5910	.6705
.6000	.6664	.5958	.6632	.5906	.6727	.5903	.6717
.1690	.2920	.1690	.2920	.1690	.2920	.1690	.2920
.4310	.3744	.4268	.3712	.4216	.3807	.4213	.3797
Net diam. =	.8054		.7980		.8023		.8010

Our primary object in taking the contour was to find a suitable place to insert a pitot tube at which point we could also obtain the cross-section area of the jet. We would have been interested in exploring the shape of the jet at greater distances out but were prevented at the time by the limits of our calipers, since it would be necessary to adjust the instrument to allow for the downward drop of the stream for measurements much beyond 2 ins. out from the orifice plane.

On the ¼-in. orifice beyond our "least section" point for a distance of a radius the diameter remained constant with a variation no greater than 4 parts in 790. But we were not able to notice any decrease in diameter and regarded variations in our readings due more to fluctuations than to any change in the diameter of the stream. The jet issued from the ¼-in. orifice smooth and perfectly transparent (which in itself seems sufficient evidence that the velocity is constant throughout the section) and remained so for a distance of many feet from the orifice as the view will show (see Fig. 2). The larger orifices, especially the 2 and 2.5-in., showed a perceptible spreading out of the jet at very much less than a radius' distance beyond the least section point as we located it. The largest jets showed also a much more roughened surface than did those from the smaller orifices.

DIAMETER AT THE LEAST SECTION.

After the contours had been made the data were inspected, the positions for the least sections were selected and the diameters at those points were taken. Later, on plotting the contour curves, it was found that the positions chosen for the least sections were further out than those indicated on the curves, but they still lay well within the least area zone where the diameter appeared to be constant for some distance.

Five readings were taken for each diameter; and diameters at four different positions 45° apart were found for each orifice at nine different pressures ranging from 3 to 40 lbs. static pressure. Table VI. is a sample page from the observed readings for the 1-in. orifice and includes readings observed with 5-ft., 40 lbs. and 100 lbs. static pressures.

The coefficient of contraction for each orifice was computed from the averages of 36 diameters, as shown in Table VII. From Table II. the following constants are taken:

Orifice, ins.....	0.75	1	1.5	2	2.5	Average
Coef. Cont[g]....	0.6134	0.6115	0.6051	0.6082	0.5955	0.60674
Coef. Discharge.	0.6111	0.6097	0.6085	0.6083	0.5956	0.60664

From the average results, the coefficient of velocity equals:

$$\frac{\text{Coefficient of discharge}}{\text{Coefficient of contraction}} = \frac{0.60664}{0.60674} = 0.99983.$$

In finding the coefficient of velocity experimentally the method recommended in text-books and most frequently used has been to measure the coordinates of the path of the jet and to compute from them the values of the coefficient. Values[g] thus obtained have ranged from 0.97 to 0.998. Bovey[h] by this method has obtained an average value of 0.99 for orifices of ¼, ½, 1-in. diameters and under heads from 6 to 20 ft. Our results for the ¾-in. and 1-in. orifices seem to agree very well with Professor Bovey's results.

TABLE VII.—Diameters at Least Section for Various Angles and Heads.

Size.	Head, lbs.	Head, ft.	Horizontal.	45° up.	45° down.	Vertical.	Average
.751-in.	3	6.93	.5926	.5869	.5877	.5885	.5889
	5	11.55	.5918	.5869	.5924	.5906	.5904
	7.5	17.32	.5924	.5834	.5886	.5903	.5887
At .8-in.	10	23.09	.5912	.5873	.5873	.5896	.5889
out from	15	34.64	.5900	.5864	.5840	.5870	.5868
plane of	20	46.19	.5901	.5870	.5834	.5872	.5870
orifice.	25	57.74	.5899	.5846	.5873	.5936	.5889
	30	69.28	.5886	.5829	.5855	.5902	.5868
	40	92.38	.5911	.5852	.5837	.5881	.5870
		Av..	.5909	.5859	.5867	.5883	.5882
1.025-in.	3	6.93	.8054	.7992	.7976	.7948	.7989
	5	11.55	.8048	.8022	.7978	.7862	.7978
	7.5	17.32	.7993	.8050	.7934	.8052	.8007
At .9-in.	10	23.09	.8090	.8048	.7970	.8008	.8022
out from	15	34.64	.8048	.7956	.8034	.8024	.8016
plane of	20	46.19	.8095	.8002	.8004	.8036	.8027
orifice.	25	57.74	.8053	.8033	.8045	.8025	.8039
	30	69.28	.8045	.7986	.7962	.8064	.8014
	40	92.38	.8083	.8015	.8037	.8028	.8048
		Av..	.8049	.8012	.7996	.8006	.8015
1.507-in.	3	6.93	1.1727	1.1680	1.1650	1.1674	1.1687
	5	11.55	1.1693	1.1677	1.1687	1.1713	1.1670
	7.5	17.32	1.1659	1.1715	1.1647	1.1708	1.1682
At 1.2-	10	23.09	1.1649	1.1707	1.1831	1.1819	1.1751
in. out	15	34.64	1.1693	1.1787	1.1735	1.1781	1.1749
from	20	46.19	1.1693	1.1775	1.1717	1.1803	1.1762
plane of	25	57.74	1.1711	1.1765	1.1691	1.1809	1.1744
orifice.	30	69.28	1.1671	1.1729	1.1717	1.1781	1.1725
	40	92.38	1.1669	1.1760	1.1763	1.1731	1.1733
		Av..	1.1675	1.1731	1.1716	1.1764	1.1722
1.9975-in.	3	6.93	1.5510	1.5548	1.5883	1.5423	1.5591
	5	11.55	1.5529	1.5513	1.5533	1.5619	1.5543
	7.5	17.32	1.5526	1.5527	1.5508	1.5620	1.5545
At 1.5-	10	23.09	1.5553	1.5487	1.5613	1.5569	1.5556
in. out	15	34.64	1.5671	1.5565	1.5525	1.5557	1.5580
from	20	46.19	1.5593	1.5597	1.5539	1.5565	1.5574
plane of	25	57.74	1.5575	1.5565	1.5621	1.5691	1.5613
orifice.	30	69.28	1.5623	1.5541	1.5615	1.5675	1.5614
	40	92.38	1.5582	1.5690	1.5562	1.5677	1.5605
		Av..	1.5571	1.5527	1.5611	1.5599	1.5581
2.500-in.	3	6.93	1.9315	1.9319	1.9167	1.9117	1.9229
	5	11.55	1.9138	1.9285	1.9219	1.9169	1.9203
	7.5	17.32	1.9185	1.9197	1.9233	1.9257	1.9281
At 1.8-	10	23.09	1.9215	1.9337	1.9519	1.9243	1.9328
in. out	15	34.64	1.9497	1.9387	1.9331	1.9199	1.9353
from	20	46.19	1.9401	1.9407	1.9295	1.9163	1.9294
plane of	25	57.74	1.9327	1.9379	1.9263	1.9215	1.9296
orifice.	30	69.28	1.9523	1.9541	1.9341	1.9215	1.9405
	40	92.38	1.9357	1.9447	1.9321	1.9191	1.9329
		Av..	1.9329	1.9377	1.9289	1.9196	1.9295

EFFECT OF INCREASE IN STATIC PRESSURE ON THE LEAST SECTION AREA.

From an examination of the diameters at the least section given in Table VII. it will be seen that there is no regular variation in the average diameters taken at the four different positions, namely: the horizontal, 45° above the horizontal, 45° below the horizontal, and the vertical. For the ¾ and 1-in. orifices the horizontal diameter is the largest; for the 1½-in. the vertical diameter is the largest; for the 2-in. the 45° down is the largest; while for the 2.5-in. the 45° up is the largest. Hence to determine the change in the diameter due to increase in static pressure averages for all four positions were taken for each static head or pressure. These averages

[g] Merriman's "Hydraulics," 1903, p. 113.
[h] Bovey's "Hydraulics," 1904, p. 32.

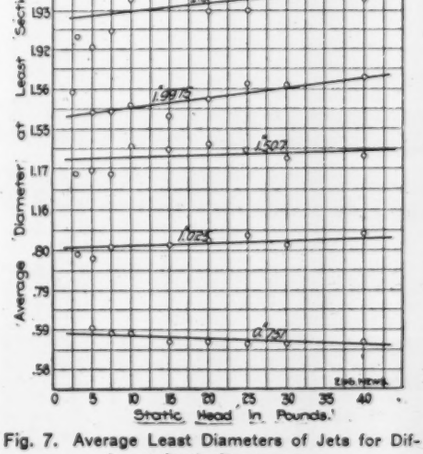


Fig. 7. Average Least Diameters of Jets for Different Static Pressures.

show for the ¾-in. orifice a tendency toward a decrease as the pressure increases. For the other orifices the average diameter seems inclined to increase slightly as the pressure increases. Fig. 7 brings this out better. Curves were plotted showing the diameters for each orifice at different static pressures. Selecting from these curves the diameters for 5 lbs. and 40 lbs. and comparing we find the following per cent. of variation in the coefficient of contraction:

Orifice, ins.	0.75	1	1.5	2	2.5
Diam. for:					
5-lb. static head	0.5890	0.8010	1.1725	1.5540	1.9285
40-lb. static head	0.5865	0.8030	1.1745	1.5625	1.9365
Coef. Cont. 5-lb. head	0.6151	0.6107	0.6054	0.6050	0.5953
Coef. Cont. 40-lb. head	0.6094	0.6137	0.6075	0.6115	0.6000
Average Coef. Cont.	0.6123	0.6122	0.6065	0.6083	0.5977
% variation from Average coefficients of contractions=0.6075.	0.0093 % Decrease.	0.0049 % Increase.	0.0034 % Increase.	0.0107 % Increase.	0.0079 % Increase.

With the exception of the ¾-in. orifice the curves show a slight increase in diameter with increase in pressure. For the larger orifices the plotted results show a greater variation in the readings so that it is difficult to tell just how much increase, if any, is due to increase in pressure.

An average of the per cents. variation, including ¾-in. orifice, gives 0.0035% increase.
An average of the per cents. variation, without ¾-in. orifice, gives .0067% increase.
Average coefficients of contraction as computed from data equals 0.60674.
Average coefficient of contraction from data by means of the curves equals .60750.
The per cent. of variation equals 0.1%.

The static head on the 1-in. orifice was increased to 100 lbs. and diameters were taken at the least section. The average results are given.

Results taken from least diameters on 1-in. orifice.

Static head, lbs.	Av. diam.	Coef. Per cent. contraction.	variation from Coef. Cont. for 5 lbs.
5	0.8010	0.6107	.....
40	0.8030	0.6137	0.0049 increase.
100	0.8017	0.6117	0.0016 increase.

For 100 lbs. the coefficient of discharge $= \frac{q}{\sqrt{2gh}} =$

$\frac{0.0427 \text{ (from the curve)}}{\sqrt{2gh}} = 0.6113.$

For 100 lbs. the coefficient of velocity $= \frac{0.6113}{0.6117} = 0.9993.$

These results indicate for an increase of pressure to 100 lbs. only a slight variation which probably lies within the errors of observation. For the 1-in. orifice at 100 lbs. pressure the velocity of approach in the drum did not exceed 0.25 ft. per sec.

THE VELOCITY IN THE LEAST SECTION FOUND BY THE PITOT TUBE.

The jets were traversed at the least section with a pitot tube to determine the velocity in the cross-section. This also gives another method of ascertaining the coefficient of velocity. The pitot tube used in traversing the jet was made of ¼-in. seamless brass tubing with the end cut away in the inside forming a bell-mouth tip. This form of tip was chosen because it seemed to be the easiest to make, although previous experiments in the laboratory[i] have shown that the form of the end of the tip makes little or no difference, the essential point being to have the plane of the tip at right angles to the direction of the current. In his experiments on the pitot tube, Mr. M. W. White[j] has reached the same conclusions.

To make the proper attachment to our hydraulic caliper carriage, for traversing the jet, we had to bend the pitot in the form of a goose neck (see Fig. 2 "D"). The end of the pitot was cylindrical and perfectly straight a distance of 1 in.

The difference in head between the static head and that shown by the pitot tube, or the velocity head, was indicated on a differential gage.

At first we tried to measure the heads separately by means of two mercury columns about 7 ft. long. There was too much vibration for close reading, so we attached a water manometer directly to the gage connection and to the pitot which indicated the difference of level by an entrapped air column. This gave no appreciable difference between the two heads; in fact it was extremely hard to tell which was the higher.

Finally the most satisfactory fluid for the differential manometer (see Fig. 3 sketch "E") was found to be made up of a mixture of equal volumes of tetra-chloride of carbon and benzine, which gave by determination a specific gravity of 1.1.[k] This used in a manometer with the upper part completely filled with water, gives an instrument magnifying ten times.

At the beginning of a traverse the pitot tip was brought in contact with the edge of the jet and then was moved until the internal area ($^3/_{16}$-in.) of the tip was just inside the jet (see Fig. 3, Sketch F). The tip was then advanced by increments of 0.15-in. along the horizontal diameter until the outer edge of the jet had been reached. This was known both by the sudden change in the pitot reading and also by comparing the distance traversed with the diameter at the least section as previously determined.

Table VIII. gives the results of the average pitot tube determinations. The manometer readings represent the excess of the static head over that of the pitot or velocity head. The ratio of pitot to the static head is given; and also the square root of the ratio which is equal to the ratio of the velocity by the pitot to that corresponding to the static head, or the coefficient of velocity. Our readings show very little difference between the two heads. Our manometer being so sensitive, fluctuated somewhat in response to the slight pulsation due to the pump. Yet we think we are correct within ½-in. on the manometer or 0.05-in. of water, which for the lowest heads of 3 lbs. affect our results in the fourth place only.

We would conclude that the coefficient of velocity by the pitot was 1.0, within an eligible fraction of 1%. As already shown, the coefficient of velocity, figured from the average coefficient of discharge by calibration and from the average coefficient of contraction by measurement, is found to be 0.99983. This compares very well with the results with the pitot tube (av. 0.99993) and would seem to indicate that the constant of the pitot tube was also unity. This agrees with other experimenters with the pitot tube. Prof. S. W. Robinson[l] using the pitot for measuring the flow of air, in 1886 and earlier, and Mr. J. R. Freeman,[m] in 1880, for the flow of water from nozzles, both found the pitot constant to be unity. Likewise, Mr. W. M. White[n] in his investigation of the pitot formula and Prof. W. B. Gregory[o] in connection with the tests of hydraulic dredges for the Mississippi River Commission, found that a properly constructed pitot had a constant of one.

CONCLUSIONS.

The results of our experiments may be summed up as follows:

(1) The coefficient of discharge decreases slightly as the size of the orifice increases.

(2) The distance from the plane of the orifice to the least section decreases as the size of the orifice increases, but less rapidly as the size increases.

(3) The coefficient of contraction decreases as the size of the orifice increases.

(4) The increase in static pressure seems to have little effect on the area at the least section, where there is no velocity of approach.

(5) The velocity is found to be uniform throughout the section and is equal to that corresponding to the static pressure within a negligible fraction of 1%.

(6) The constant of the pitot is unity.

(7) The coefficient of velocity is unity within a small fraction of 1%.

(8) The pitot tube will give the velocity head due to the velocity of approach in addition to that corresponding to the static head, and if there is a velocity of approach the diameter at the least section would have to be determined since there may be an increase in the area at the least section.

This point we intend to investigate more fully as soon as time permits, for we believe that, by using the pitot tube and knowing the diameter at the least section, there is available a simple and accurate form of water meter which often times might be used with little expense and trouble in fitting it up, where other devices for measuring the flow of water could not be so conveniently used.

[i] Boyd and Judd, Eng. News, March 31, 1904.
[j] Journal of the Association of Engineering Societies, August, 1901.

[k] See Photo-Pitotmeter, E. S. Cole, Trans. Am. Soc. C. E., April, 1902, Vol. XLVII.
[l] Prof. S. W. Robinson, "Van Nostrands's Engineering Magazine," Aug. 1, 1886.
[m] J. R. Freeman, "Trans. Am. Soc. C. E.," 1889, Vol. XXI.
[n] W. M. White, "Jour. Assoc. Eng. Socs.," August, 1901.
[o] W. B. Gregory, "Trans. Am. Soc. M. E.," December, 1904.

TABLE VIII.—PITOT TUBE READINGS.

.751-in. Orifice.									
Head, in lbs.	3	5	7.5	10	15	20	25	30	40
Manometer	.55	.50	.55	.45	.50	.60	.67	.90	.95
Inches water	.055	.050	.055	.045	.050	.060	.067	.090	.095
Ratio Pitot to static head	.9994	.9996	.9997	.9999	.9999	.99997	.9999	.9999	.9999
√Ratio or coefficient of velocity	.9996	.9998	.99997	.99997	.99994	.99999	.99995	.99995	.99996
1.025-in. Orifice.									
Manometer	.07	.40	.56	.56	.44	.29	.73	.66	.68
Inches water	.007	.040	.056	.056	.044	.029	.073	.066	.068
Ratio Pitot to static head	.99991	.9997	.9996	.9998	.9998	.99996	.9999	.99991	.99996
√Ratio or coefficient of velocity	.99996	.99988	.9998	.9999	.99993	.99999	.99994	.99996	.99998
1.507-in. Orifice.									
Manometer	.21	.26	.30	.30	.44	.42	.90	1.18	.36
Inches water	.021	.026	.030	.030	.044	.042	.090	.118	.036
Ratio Pitot to static head	.9997	.9997	.9999	.9999	.99986	.99993	.99986	.99986	.99997
√Ratio or coefficient of velocity	.99988	.99988	.99994	.99996	.99993	.99997	.99993	.99993	.99999
1.9975-in. Orifice.									
Manometer	.40	.40	.35	.30	.25	.34	.46	.36	.61
Inches water	.040	.040	.035	.030	.025	.034	.046	.036	.061
Ratio Pitot to static head	.9996	.9997	.9997	.99991	.99994	.99993	.99993	.99996	.99994
√Ratio or coefficient of velocity	.9998	.99988	.99988	.99996	.99998	.99997	.99997	.99998	.99998
2.500-in. Orifice.									
Manometer	.16	.35	.35	.30	.45	.15	.10	.50	.33
Inches water	.016	.035	.035	.030	.045	.015	.010	.050	.033
Ratio Pitot to static head	.9998	.9997	.9997	.99991	.9999	.99995	.99998	.99994	.99997
√Ratio or coefficient of velocity	.9999	.99988	.99988	.99996	.99995	.99998	.99999	.99998	.99999

STREET TRAFFIC IN NEW YORK CITY, 1885 AND 1904.*

By Clifford Richardson, Assoc. Am. Soc. C. E.†

In December, 1885, Francis V. Greene, M. Am. Soc. C. E., read a paper before this society, entitled "An Account of Some Observations of Street Traffic,‡ in which he presented certain data in regard to ten of the large cities of the United States, collected in that year by the employees of the Barber Asphalt Paving Co. He also called attention to the desirability of having traffic measured systematically at frequent intervals, on a uniform system, in the leading cities of the world, for comparative purposes. Since that date no further observations have been made, as far as the writer is aware, in any of the cities considered by Mr. Greene, either in the United States or abroad, with the exception of a count, made in 1896, of the number of vehicles traversing Fifth Ave., New York City, at various points, before the removal of the granite block pavement from that street.

In November and December, 1904, additional data were collected, at the writer's suggestion, by employees of the Barber Asphalt Paving Co., on ten streets in New York City, for the purpose of determining the traffic carried by a number of representative streets at that time. The results of this count, as worked out by employees of the company, were presented not long after in the engineering journals§ in comparison with the data of 1885, and with data for Paris and London cited by Mr. Greene. The weights attributed to the vehicles, by those working up these data, were made to include the horse or horses, but this was not done in calculating the tonnage in Mr. Greene's paper, and has not been the practice abroad. It seems, therefore, that the comparisons made at that time were not correct. All the data having since then come into the writer's possession, it has seemed worth while to compare the results derived from them with those obtained in 1885, although they have also been worked out on a basis which, it is believed, represents more accurately the traffic carried by the streets in 1904.

For the purpose of determining the tonnage carried by the various streets, the writer has assigned the following weights to the different classes of vehicles, from data furnished by several of the most prominent carriage and truck builders:

Vehicle.	Weight, lbs.
Cab	1,050
Carriage	1,575
Automobile, electric	4,000
Automobile, gasoline	2,325
Electric truck	5,000
One-horse delivery wagon	1,000
Two-horse delivery wagon	1,500
Omnibus (Fifth Ave.)	3,200
Omnibus (streets other than Fifth Ave.)	1,550
Trucks	2,000 to 16,000

On this basis, the total tonnage per foot of width of street has been calculated, excluding the width of car tracks, where these exist, because the main traffic, especially on asphalt, is confined to that portion of the street lying between the tracks and the curb.

From the observations of 1885, Mr. Greene divided the traffic into only three classes: light weight (less than 1 ton), medium weight (between 1 and 3 tons), and heavy weight (more than 3 tons). For the purpose of comparison with the data of 1885, the observations of 1904 have also been calculated on this basis, using, however, more detailed data in regard to vehicles which weigh more than 3 tons, as those of 6 and 7 tons are not uncommon now in the streets of New York.

Having the preceding data, a comparison of the traffic of 1885 with that of 1904 can be made satisfactorily only on Fifth Ave., the other points of observation in the two years not being the same. It appears that on

*Abstract of a paper presented at the meeting of the American Society of Civil Engineers, New York, Sept. 19, 1906.
†New York Testing Laboratory, Long Island City, N. Y.
‡Trans. Am. Soc. C. E., Vol. XV., p. 123.
§See Eng. News, Feb. 21, 1905.

Fifth Ave. 5,460 vehicles passed the Worth Monument in 1885, and 6,300 passed over the granite pavement in 1896; whereas, in 1904, 12,068 were observed between 33d and 34th Sts., an increase of about 120%. In 1885 Mr. Greene estimated the average weight per vehicle as 0.68-ton. In 1904, according to the weights which have been assumed as correct, the average weight per vehicle was 0.82-ton, or 0.83-ton on the former basis. In 1885, 91% of the vehicles weighed less than 1 ton, while, in 1904, 79% (a much smaller number) were of this character. The traffic on this street, therefore, while still composed largely of light vehicles, has increased to an enormous extent in eight years, and the average weight per vehicle to a very considerable extent. The tonnage per foot of width, on the Greene basis of weight, has increased from 94 to 251, or to 247 on the writer's basis.

With the exception of First Ave., at 26th St., Fifth Ave. carries the heaviest traffic in the city of New York, although this traffic is composed of the largest percentage of light-weight vehicles. This fact, however, must be given a great deal of consideration, since a small tonnage of heavy-weight vehicles will undoubtedly cause a greater deterioration of the pavement than a larger traffic of lighter ones.

In 1885 the traffic on Broadway, near Pine St., amounted to 273 tons per ft. of width. No enumeration was made at this point in 1904, but at the two points where the count was made (between 18th and 19th Sts., on asphalt, and between Franklin and Leonard Sts., on granite block, the tonnage only reached 107.2 in the one case, and 65.5 in the other. These figures are on the Greene basis of calculation, which includes the car tracks in the width of the street, and which must be used for a comparison of 1885 and 1904 data. On the writer's basis of calculation, the tonnage at these two points amounts to 169.8 and 96.9. These figures are exceeded by those for First, Fifth and Eighth Aves. and for Fourth St. It is quite surprising, therefore, to find that the Broadway traffic is not a heavy one, at the present time, as compared with that of several other streets in New York, while it has, very possibly, decreased rather than increased since 1885.

It is an interesting fact that the heaviest traffic which has been counted in New York is on First Ave. at 26th St., and that it is largely made up of vehicles of a heavy class, averaging 2.18 tons in weight, only 26% being of less than 1 ton. It is also worthy of note that the pavement on this street is not of a form of construction calculated to withstand such a traffic, as it is not supported by a hydraulic concrete base.

For the purpose of comparing the traffic on the streets of New York with that upon streets in London and Liverpool, it seems worth while to present here certain data in regard to the latter, collected by Mr. Greene from the papers of Messrs. Hayward, Deacon, Howarth and Stayton. It appears that the tonnage per foot on some streets in London was far greater even in 1873, than anything found in New York, reaching 412 on King William St., a street of the same width as Fifth Ave., where the vehicles average 1.06 tons in weight and 16,484 in number. On the other hand, on no street in London was the average weight of the vehicles so great as that on First Ave. It must be remembered that in London there is an enormous number of omnibuses which are not found on New York streets; for instance, of the 10,776 vehicles passing along Piccadilly, 32% are of this description, and average 2 tons in weight, while only 56% are cabs, averaging ½-ton each. Notwithstanding this, the traffic on Fifth Ave. approaches in tonnage per foot that on Piccadilly in 1873. Unfortunately, there are no data available for the latter street at the present time. The traffic, no doubt, has increased largely.

It has been the custom on the Continent, and, in some cases, in England, to classify the traffic according to the number of horses attached to the vehicles, or, as it is denoted in Paris, by the number of collars. The records show that the traffic in Paris in 1881 and 1882 was very much greater than that on the streets carrying the heaviest traffic in New York City at the present time. In fact, no traffic has been counted on any street in any city in the world which can equal that for the Rue de Rivoli in Paris in 1881, which at that time was nearly three times as great as that on Fifth Ave. to-day, the width of the two streets being practically the same. As in the case of some of the London streets, the great tonnage carried by this street is to be attributed to the large number of extraordinarily heavy omnibuses which traverse it, these vehicles being much heavier than those used in New York or London, and each counting for three collars. If to each collar a weight of 0.5 is assigned, the tonnage on this street would amount to 534 per ft. of width, which is nearly double that on Fifth Ave.

As has been said, according to the method adopted on the continent, in England, and by Mr. Greene in 1885, no consideration was given to the weight of the horses in calculating traffic to which the pavements were subjected from the number of vehicles enumerated. Mr. W. G. Root, District Manager of the Barber Asphalt Paving Co., in working out the data which were presented in the engineering journals in 1905, included the weight of the animals as well as that of the vehicles, in arriving at a conclusion as to the traffic carried by the various streets.

To the writer, the latter method of statement, if the estimate of the weight of the vehicle and animal is correct, seems much more desirable. From the point of view of one who has been employed for many years in the construction of pavements and in observations as to the causes of their deterioration, it has become evident that the latter is due much more to the impact of the horses' hoofs, especially in the case of asphalt pavements, than to vehicles rolling quietly over the surface; this, of course, with the understanding that the base supporting the pavement is sufficiently rigid to do so satisfactorily and prevent vibration in the wearing surface. For these reasons it seems desirable to consider the data in the accompanying table as representing the true traffic on the various streets in regard to which the data were calculated, and to suggest that, in any observations made in the future, due consideration should be given to the weight of the animal as well as to that of the vehicle.

TRAFFIC ON NEW YORK STREETS: 1904.

From 7 a. m. to 6 p. m. Weight of animals included with vehicles. Width does not include the car tracks, as these carry only a small proportion of the traffic.

Street.	Width, ft.	Average tonnage per vehicle.	Average tonnage per lin. ft. of width for 11 hrs.
5th Ave., 33d to 34th St.....	40	1.64	481.85
1st Ave., 26th to 27th St.....	44	3.18	435.58
8th Ave., 35th to 36th St.	44	2.28	296.02
Broadway, 18th to 19th St.....	25	1.97	299.63
4th St., Wooster St. to West Broadway	32	2.73	289.18
B'way, Franklin to Leonard St.	28	1.74	202.42
10th Ave., 22d to 23d St.....	34	2.69	227.45
3d St., Mercer to Greene St..	28	2.51	140.50
2d Ave., 34th to 35th St.....	46	2.24	117.86
34th St., Broadway to 7th Ave.	24	2.03	89.22

REINFORCED-CONCRETE WAREHOUSES AT SAN FRANCISCO, CAL.

In the rebuilding of the city of San Francisco after the recent earthquake and fire there seems likely to be an extensive use of reinforced concrete construction; among the first of the new structures of this class will be a group of buildings now under construction at Second and Folsom Sts. for the A. Schilling Co., tea, coffee and spice merchants. These buildings will include the following: tea warehouse, 70 × 175 ft.; coffee warehouse, 40 × 60 ft., with coffee roasting tower 40 × 40 ft.; baking powder warehouse, 60 × 165 ft.; office building, 80 × 80 ft. (two stories); power-house, 40 × 40 ft. (one story, 25 ft. high). All these are three stories high, with basements, except as noted. There will also be some other buildings about 40 × 150 ft., and a shipping court 145 × 145 ft., with concrete roof. The architect is Mr. Edmund Kollofrath, of San Francisco; and Mr. C. F. Wieland, 2003 Benvenue Ave., Berkeley, Cal., is consulting engineer. The contract has been awarded to the Continental Construction Co., of Birmingham, Ala., and the steel reinforcement system is that of the American Wire Fence Co., 189 La Salle St., Chicago.

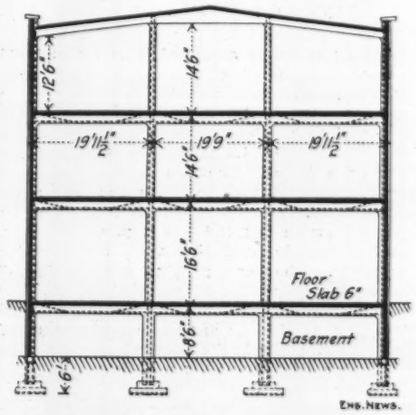


Fig. 1. Cross-Section of Tea Warehouse of Reinforced Concrete for the A. Schilling Co., San Francisco, Cal.

Longitudinal beams for floors, 10 × 12 ins., and for roof, 10 × 10 ins., all reinforced by 4 ¾-in. round rods.

Girders for floors, 12 × 20 ins., and for roof, 10 × 14 ins., all below slabs; 8 1⅛-in. reinforcing rods in floor girders and 6 1-in. rods in roof girders.

Floor slabs 6 ins. and roof slabs 3½ ins. thick, reinforced with one layer Nos. 9 and 11 wire fabric; floors also reinforced by 8 1⅛-in. rods, 12 ins. c. to c., and roof by ¾-in. rods, 20 ins. c. to c.

Columns 22 × 22 ins. in basement, and 16 × 16 ins. in first and second stories, and 10 × 10 ins. in third story, reinforced by 8 1¼-in. rods in basement and first story and 4 ¾-in. rods in second and third stories.

Pilasters 16 × 22 ins., 14 × 16, 12 × 16 and 10 × 10 ins., in basement and successive stories, reinforced same as columns.

All the buildings will be of monolithic concrete throughout; they were originally intended to be of structural steel with concrete fireproofing, but owing to the difficulty of obtaining steel the reinforced concrete system was adopted. The concrete will be composed of 1 part Portland cement, 2 parts sand and 4 parts of 1-in. broken stone. The accompanying cut, Fig. 1, giving a cross section of the tea warehouse, shows very clearly the general design and construction, together with the arrangement of the reinforcing material. The floor panels are about 14 × 20 ft.; the floors are designed for a live load of 250 lbs. per sq. ft. The column footing construction is shown in Fig. 2.

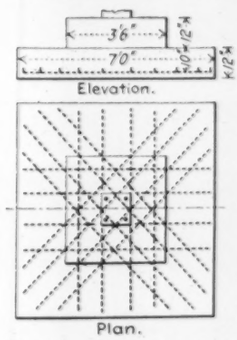


Fig. 2. Plan and Elevation of Column Footings of Reinforced Concrete Warehouse.

16 1¼-in. round rods and one layer Nos. 7 and 11 wire fabric used horizontally in footing; 8 rods used vertically in column.

The steel reinforcement consists of round rods supplemented by the wire netting of rectangular mesh (4 × 6 ins.) which is made for this purpose by the American Wire Fence Co., of Chicago. The rods and wire are all of high-carbon steel; the No. 9 wire is said to have an elastic limit of 118,000 lbs. and an ultimate breaking strength of 155,000 lbs. per sq. in. In the netting, the longitudinal and cross wires are securely locked together; that used in the floors is of No. 9 and No. 11 wire. The netting is made in widths of 3 to 5 ft., and in lengths of about 175 ft., being put up in rolls for shipment. When it is set in place the adjacent layers are connected by hooks so as to make practically a continuous sheet both longitudinally and transversely. This fabric is used not only in the floors and roof, but also for wrapping all the columns, beams and girders.

This system of reinforcement has already been used in a number of important buildings in other cities, including the extensive car shops of the Twin City Rapid Transit Co., at Minneapolis, Minn., the State Capitol at Topeka, Kan., and the terminal railway station at Mobile, Ala.

A MONO-RAIL SYSTEM is now in successful operation for mine haulage at Langlaagte Deep, South Africa. About 3,000 ft of road are equipped with the system, which is used on development work, both driving and winzing, and in tramming both in the stopes and from the stope boxes. The equipment consists of vertical iron hangers firmly secured in the roof of the drive, winze, or stope, with a right-angle bend 6 ins. from the bottom. This bend in each hanger forms a step on which is placed a 16-lb. rail. The hangers are made in two parts to permit of adjustment in bringing the rail to grade, and the upper part of the hangers where they are fastened to the roof is made in two forms, depending upon whether for drives and cross-cuts or for winzes and stopes. The distance between hangers averages about 8 ft. The trucks consist of a U or V-shaped body hung at each end by a single pin to a bridle which is attached by two vertical drawbars to a pair of two-wheeled bogies. In winzes and very narrow stopes, the wheels, of which there are four in tandem on each truck, are 6 ins. in diameter; in wide stopes they are 8 ins., and in the drives 8 to 12 ins. The motion is imparted by winding, which is done by hand for short distances and by an air winch for greater distances. The advantages of the mono-rail system over the ground-level system for tramming are, less interruptions on the track and less friction owing to there being no spill on the rail in filling the trucks at the stope boxes; less liability of the trucks getting off the track; a cleaner and quicker tip of the truck body; and greater protection of the rails and wheels by being high and dry.

A NEW CENTRIFUGAL PUMP DESIGNED FOR USE WITH A WIDE RANGE OF HEADS.

The centrifugal pump shown in Figs. 1 and 2 was recently designed by Mr. E. S. Lea and Mr. Julian Degen, of the Lea-Degen Pump Co., Trenton, N. J. The designers aimed to secure high efficiency and a low cost of manufacture, the latter by designs that would enable one set of patterns to be used for a large range of heads.

The pump consists essentially of two shrouded wheels, mounted on the same shaft in a double case. The case is so partitioned that the water is drawn from the source of supply and put under pressure by the first wheel, and then delivered to the suction chamber of the second wheel. The second wheel then imparts to the water the same amount of energy it receives from the first wheel, thus increasing the pressure, and then delivers the water into a spiral discharge conduit which terminates in a diverging nozzle connecting with the main pipe.

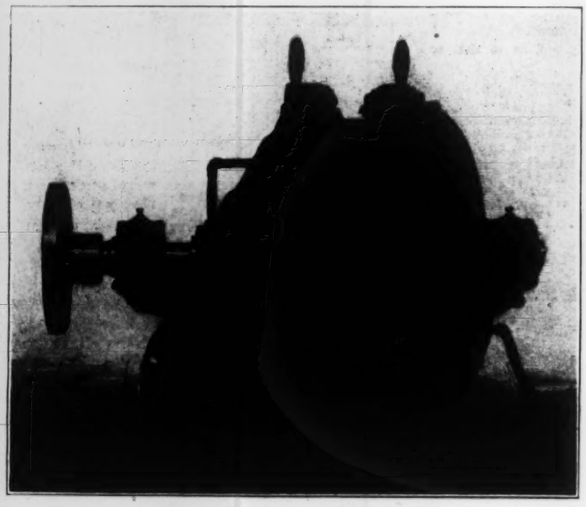


FIG. 1. EXTERIOR VIEW.

FIG. 2. INTERIOR VIEW

TWO-STAGE 10-IN. CENTRIFUGAL PUMP FOR VARYING HEADS.

Messrs. E. S. Lea and Julian Degen, Inventors.

SPECIAL FEATURES OF THE PUMP.

The special features of the pump are as follows:

1. The case is divided through its horizontal diameter by bolted flanges so that its top half can be quickly freed and lifted off, without disturbing either suction or discharge connections, thereby affording easy access to the internal parts of all the stages at once.

2. By means of bolted circumferential divisions of the case, provision is made for either using the suction and discharge end of a case together as a single-stage pump, or for adding as many intermediate sections as may be necessary to afford any desired pressure at any fixed speed. Additional stages can therefore be installed after a pump has been in operation without wasting any parts of the existing case.

3. A special arrangement of double cup-leather packing is used for both the suction and discharge ends and for the intermediate sections. The cup leathers are held against a flat collar, on extended pump-wheel sleeves, in such a manner that the leathers can follow up as they or the collars are worn, or the shaft may be shifted at will in either direction, with the leathers following, without changing the location of the leather with relation to the shaft collar, thus making a practically water-tight joint at all times. A spiral spring is used between each pair of cup leathers, to ensure their seating against the collars before pressure is put on the pump. Provision is made for setting out the leather packing on the suction end of the shaft by the water pressure of the high side of the pump.

4. On the outside of the main bearings, at each end, is placed a ball thrust-bearing with adjusting collars, for shifting the shaft endways to balance the end thrust of the pump runners. The balancing is accomplished by means of variation in the width of water space, on both sides of the wheels between the rim of wheel and the case. Experiments have shown that as the wheel was moved laterally in the case, the pressure between the wheel and case increases on the side where the clearance is greatest, and is reduced on the opposite side.

TESTS CONDUCTED ON THE PUMP.

A very complete series of capacity and efficiency tests were conducted on the pump by Prof. J. E. Denton, of the Stevens Institute of Technology. The pump was driven by a General Electric direct-current generator of 220 volts and 385 amperes capacity, used as a motor. The two machines were directly connected together. As the pump was designed for practically equal efficiencies for speeds between 400 and 600 r. p. m., efficiency tests were made at 400, 500 and 600 r. p. m., respectively. The accompanying curves, Figs. 3, 4 and 5 have been plotted from the data obtained.

No data were recorded at speeds varying more than two revolutions from the assigned speed. For obtaining the brake horse-power, the pump was disconnected from the motor and the power of the latter absorbed by a Prony brake over the same range of watts applied to drive the motor during the pump tests. The horse-power corresponding to the watts applied to drive the motor during the pump tests was thus determined and taken as the horse-power required to drive the pump.

CONCLUSIONS.

The tests show that the pump gave the following results under conditions of maximum efficiency.

At 400 revs. 77.7% eff. with a cap. of 2,296 gals. under 43.6 ft. lift.
At 500 revs. 77.6% eff. with a cap. of 2,794 gals. under 67.4 ft. lift.
At 600 revs. 77.97% eff. with a cap. of 3,235 gals. under 100.7 ft. lift.

In round numbers, therefore, the capacity at maximum efficiency is directly proportional to the revolutions, and the lift, or head, is proportional to the square of the revolutions. At each speed the efficiency averaged more than 76% over a range of 600 gals. of capacity for the two lower speeds, and 900 gals. at the higher speed, the head remaining constant.

It is interesting to note that Messrs. Lea and Degen can make 150 sizes and capacities of these pumps from only ten sets of patterns, thereby greatly reducing the cost of their manufacture.

OVER 98% OF THE FREIGHT CARS of the Pennsylvania Railroad lines east of Pittsburg and Erie are equipped with air brakes, according to the report made to the Interstate Commerce Commission, in compliance with the recent order of the Commission directing the common carriers of the country to report the number of freight cars owned and the number equipped with air brakes. (Eng. News, Aug. 23, 1906, p. 213.) The total number of freight cars owned by the Pennsylvania Co. Aug. 1, 1906, was 133,411, the total number equipped with air brakes 130,915; thus leaving 2,496 without such equipment. The company, moreover, now refuses to receive cars from other roads, and sends none of its own cars over such roads unless they are equipped with air brakes.

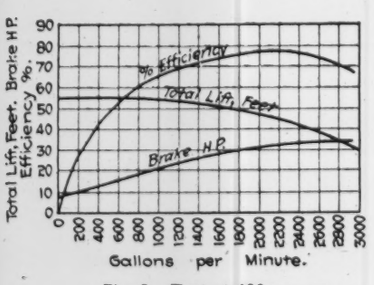


Fig. 3. Test at 400 r. p. m.

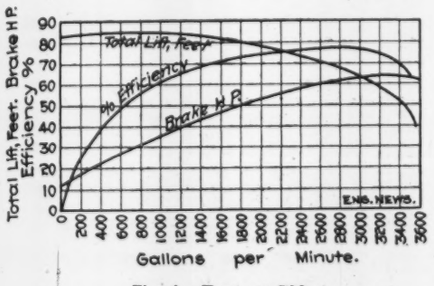


Fig. 4. Tests at 500 r. p. m.

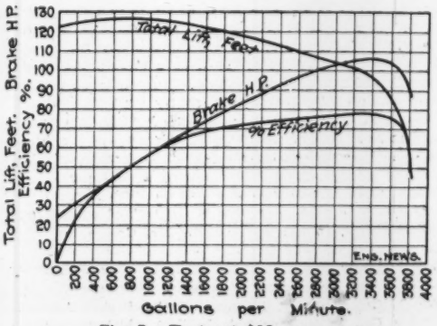


Fig. 5. Tests at 600 r p. m.

FIGS 3, 4 AND 5. CAPACITY, EFFICIENCY, AND BRAKE HP. CURVES OF LEA-DEGEN PUMP RUNNING AT VARIOUS SPEEDS AND HEADS.

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The difficulties attending the smelting of iron ore containing large quantities of fine material, together with the cost of producing pig from such material, has been the cause of many ingenious inventions during the last few years. The process, which has, perhaps, given the most satisfactory results up to the present time, is doubtless the briquetting of the fine ores, for which there have been developed many styles of apparatus.

As described elsewhere in this issue, a process has lately been developed which does away with many of the inconveniences of former methods, while at the same time it reduces the sulphur and phosphorous content of the raw ore. This process has been in continuous operation for the last year producing nodules, from fine ore, which have been successfully smelted in blast-furnaces without the difficulties attending the smelting of ore, when large quantities of fine materials are present. As described in the article referred to above, the fine ore or dust is placed in a rotary kiln with a small percentage of binder, such as tar or other compound, which is reducing in character, adhesive at low temperature, and volatile at moderate temperature. The fine ore is thus cohered into masses, which by the rotary action of the kiln are broken up into lumps or nodules, and as these lumps pass through the kiln the tar combines with the sulphur and both are volatilized. When the lower or hottest end of the kiln is reached incipient fusion occurs, the temperature being about 2,000° F., and the nodules are permanently cohered without the presence of an extraneous binder. This process has already been successfully applied to materials fusing at widely ranging temperatures, while experiment has shown that material containing volatile metals can be nodulized by this process, practically without loss of any of the valuable volatile constituents.

It is interesting to note that the central idea in this process is the use of the rotary kiln, and that thus the machine which was originally developed for and has so revolutionized the cement industry, promises to find a wide application in metallurgical work.

The relation of traffic to wear of street paving is a matter of very great importance as regard the economy and suitability of different kinds of paving material under varying conditions of traffic. Singularly enough, however, the subject has been almost neglected in America. There have been, it is true, a few investigations into the amount of traffic on city streets, but these have not included the much more important matter of the relation of this traffic to the wear of the paving on the particular streets. This matter was referred to by Mr. John W. Alvord, M. Am. Soc. C. E., in a general discussion at the last annual meeting of the Illinois Society of Engineers and Surveyors, as follows:

> The whole matter brings up that interesting question: What is the life of a pavement under any given conditions? I have asked a great many engineers what the life of a pavement was, and they all say, "It depends on the traffic." Well, of course, it does. But I never could get anybody to give me any definite idea of what the life of a pavement was under any given traffic, and I have wondered why we do not get a little further, and find out what the traffic was. If we should find what was the traffic on any given number of pavements, and find out the amount of expenditure for maintenance and the period of renewal, we would be nearer knowing what our pavements are like in comparative cost. We could then figure out the life of depreciation, as well as the first cost, and each element of cost compared to the first cost. I got up some questions for a civil service examination some time ago, and then wondered if I could answer them myself, although they were simple questions after all. I cited a macadam pavement, with a traffic of ten teams a day, and asked what would be its life; then I asked what would be the life of that same pavement with a traffic of 100 teams a day. It appears to me that we should know something about the difference between a pavement under those two conditions. But we do not know much, if anything, about it.
>
> The French do these things better than we do. All their roads throughout the country are classified, and each year they have a traffic census lasting 10 or 15 days, in which an account of the traffic over any given street or highway is made. Not only is this traffic kept and recorded, but the thickness of the roads is carefully measured. That is to say, they make cuts into the road pavement to estimate the stone that is worn off in the course of the year; and, figuring very carefully, from a large number of examples, they find out how much of the pavement wears off with a certain amount and kind of traffic, and they can proportion the allotment of money to the road in proportion to the wear of the pavement. This is done systematically and carefully, and as a result every dollar they spend is well spent and spent in places where it is needed.
>
> I wish we could get it through our heads that this matter of accumulating data is extremely important. Let us find out how much traffic passes over certain streets, in amount of tons, and find out what certain number of tons means a certain amount of wear. Then we would be in condition to study depreciation, and to compare one pavement with another, and find out what the whole cost is; not the first cost only, but the first cost plus the depreciation and the interest. And when we know that, we shall find out that some pavements which have short lives are not uneconomical, and that other pavements may be of long life and yet not be economical.

Following the suggestions contained in Mr. Alvord's remarks we made enquiry of the city engineers of a large number of cities, but found not one of them able to furnish any information on the relation of traffic to wear of pavements of different materials. Several of them stated the kinds of paving they preferred for different classes of streets, and some gave the average life of paving. Notes of this information are given elsewhere in this issue. The most extensive investigations as to street traffic in this country are those made in several large cities in 1885 and again in 1904 under the direction of the Barber Asphalt Paving Co. The later investigations form the subject of a paper by Clifford Richardson, before the American Society of Civil Engineers, an abstract of which is given in another column. It is to be noted, however, that each of these papers deals chiefly with a count of vehicles and an estimate of the weight carried; very little is said as to the character, age or condition of the paving, and nothing at all as to the rate of wear. Asphalt and stone block paving seem to be the only kinds of pavement covered. It is not an easy matter to obtain definite statistics or information as to traffic and wear of streets, but at the same time if the investigation could be taken up systematically and thoroughly in different cities there is no doubt that some valuable information might be obtained on the line suggested in the remarks by Mr. Alvord.

REINFORCED CONCRETE IN THE SAN FRANCISCO FIRE.

The recently published book on the San Francisco fire and its lessons, by Mr. A. L. A. Himmelwright, M. Am. Soc. C. E., has already been reviewed in our Engineering Literature Supplement for the current month.* The matter presented in this book, however, is of so much interest and importance at the present time and bids fair to become the subject of such vigorous discussion that we deem it well worth while to again bring it forward here.

Mr. Himmelwright's book is bound in red; and for a long time to come, the sight of a copy of this book will doubtless affect several sections of the fireproofing interest in much the same manner that a display of red affects an angry bull or a militant turkey gobbler.

As we have already stated in our review, Mr. Himmelwright gives far and away the most thorough presentation of the effects of fire on the important structures of the San Francisco burned district that has anywhere appeared. No less than 80 buildings are described in detail as to their design and construction and the condition in which they are left by the fire and earthquake; and a profusion of finely reproduced photographs enables the reader to form some opinion of his own as to the behavior of the different materials undergoing this severe ordeal. In the concluding pages of the book, the author sums up what he believes to be the lessons of the fire. It is here that his views on certain matters are sure to come in sharp conflict with those identified with some of the types of construction which receive his criticisms.

New fuel is added to the fiery controversy over the relative merits of burned clay and concrete as fireproofing materials by Mr. Himmelwright's statement: "it is apparent that hollow tile blocks sustained much greater injury under the same conditions than concrete," particularly as he follows this up by a detailed statement of the respects in which hollow tile construction proved inferior.

Undoubtedly the thing of greatest interest in Mr. Himmelwright's conclusions, and the one which will excite the greatest controversy, is his condemnation of reinforced concrete. The construction which he prefers above all others as best suited to resist either severe fires or earthquake shocks is standard steel-skeleton frame construction having columns and girders protected by concrete, the floors of segmental concrete arches of short span filled in between the beams, and with the partitions and perhaps curtain walls as well of reinforced concrete.

The building constructed of reinforced concrete throughout, however, Mr. Himmelwright condemns, and he devotes several pages at the end of his book to specifying reasons why he does not believe in this type of construction. While there are many bristling opportunities for controversy elsewhere in the book, it is over this question of the reliability of reinforced concrete that the fiercest battle is likely to be waged.

To explain now in some detail the ground on which Mr. Himmelwright bases his attacks on reinforced concrete: he does not question the strength of a properly designed and constructed reinforced concrete building to withstand all ordinary loads and service. His argument is first, that unequal settlement of the column footings will strain the concrete connections between columns and girders much more than it would the connections of a steel skeleton frame, and second and most important, that when exposed to severe fire, the steel reinforcement near the under side of beams and girders is heated, expands and weakens, with the result that the beam loses its strength.

In view of the great number of reinforced concrete buildings now under construction, and the rapidly increasing popularity of this type of construction, it is important that attention be given to this criticism. Are these buildings an engineering mistake? Are the engineers and architects and contractors who are now putting them

*Page 294, Engineering News, Sept. 13, 1906.

up, building wisely and safely for themselves and for the owners; or may they look back some day—when their work has been tried by fire—and be filled with sorrow and regret because of destruction of life and property? Surely these questions are momentous ones and worthy of serious consideration.

In dealing with this matter, it should be said in the first place that expert designers in reinforced concrete have not been wholly unaware of the danger from overheating the steel bars near the under side of concrete beams and girders in case of fire. It is evident enough that a loaded beam will fail if the tensile strength of its lower fibers is lost. If a beam of reinforced concrete is heated to a point where the rods in its under part lose their grip on the concrete and a large part of their tensile strength, failure will occur under comparatively light load. That reinforcing steel should be fireproofed with a good thickness of concrete, where exposed to fire action, therefore, is—or ought to be—understood by every designer in this field. For those who do not understand—like Mr. Kipling's lady—Mr. Himmelwright's trenchant criticism should serve a very useful purpose.

It is fair to point out here, however, that not one building constructed throughout of reinforced concrete existed in San Francisco at the time of the earthquake and conflagration. A large proportion of the fireproof buildings of steel frame construction had concrete floors, either in the form of segmental arches or flat slabs, filling the space between floor beams not over 5 to 7 ft. apart. There were also a very few buildings where wider spaces between the steel floor girders were divided by beams of reinforced concrete on which the flat floor slab rested; and it may be added that a number of these concrete slabs were reinforced. Mr. Himmelwright's criticism of reinforced concrete rests therefore on the behavior of a very few examples of this type of construction. It will be worth while, therefore, to see more particularly just what did happen in these buildings, taking our information from the data given in Mr. Himmelwright's book.

The worst case of failure of reinforced concrete beams was in the Johnson Co. building. This was a 5-story building with unprotected cast-iron columns carrying steel girders with their lower flanges exposed. Between these girders were reinforced concrete beams about 12 ins. deep spaced 5 ft. apart and carrying a 4-in. floor slab of stone concrete reinforced with ¼-in. wire cables, spaced 16 ins. apart parallel with the concrete beams and 8 ins. the other way. On top of this slab was a 22-in. deeper-fill of cinder concrete, reinforced with No. 16 hexagonal mesh poultry netting. The reinforced concrete beams were 5 ins. wide at the bottom and had 1-in. diameter round rods imbedded in them an inch from the under side. The ends of these rods were secured by nuts to the steel girders through punched holes.

In view of the unprotected columns and steel girders of this building, it is not surprising to learn that extensive failure occurred. The entire walls above the fourth story fell inward; and one whole front clear to the ground, as well as two entire bays in the interior collapsed and fell into the basement. In this basement the girders were supported on brick piers instead of cast-iron columns, and while a very hot and long continued fire raged in this basement, the floors for the most part remained intact, but were greatly deflected by the intense heat. This deflection was as much as 12 ins. in the span of 16 ft., and some of the reinforced concrete beams fell, the floor slab above them remaining in place but deflecting as much as 18 ins.

It is doubtless a fair conclusion from the experience in this building that reinforced concrete beams are less resistant to the heat of a fire than reinforced concrete in flat slabs. Nor is it difficult to see good reasons for this. The heat can penetrate a flat slab only from the under surface and it takes longer to reach and overheat the steel reinforcing members than it does where these members are near the bottom of a beam or girder, which is heated not only on the bottom but on each side.

Further, the results described in the Johnson building have been recorded also in experimental tests of reinforced concrete floors. Only last week, at a test in New York City witnessed by a member of the staff of this journal, the steel rods at the bottom of the reinforced concrete beam became overheated and separated from the concrete; but the floor with its load of 150 lbs. per sq. ft. remained in place all through the fire and the application of the hose stream that followed.

Returning now to the Johnson building, it is to be particularly noted that by Mr. Himmelwright's own account, the reinforced concrete beams in the basement were subjected to a terrific heat, reaching as much as 2,200° F. The high temperature must have been long maintained since the fire was not extinguished but burned itself out. The concrete was made with stone and not with cinders, and the steel reinforcing rods were only an inch from the bottom of the beam. Yet under these conditions, and with the bottom flange of the steel floor girders exposed directly to the fire, the floor over the basement still remained in place for the most part, except as already noted.

It seems to us that instead of this being a bad showing for reinforced concrete, that material showed a surprisingly high resistance. Mr. Himmelwright himself ascribes the fall of the upper part of the building to the earthquake, and the collapse of portions below may have been due to the exposed cast-iron columns and not to the reinforced concrete beams. While as stated above, it seems to be clearly established that reinforced concrete in the form of flat slabs resists fire better than when exposed in narrow beams or girders, yet the beams themselves appear on the whole to have made a remarkably good showing in comparison with other standard types of floor construction, such, for example, as steel beams and tile.

Particularly notable is the extraordinary deflection which some of these reinforced concrete floors showed without failing. Of course, this has been exhibited many times in tests, and engineers experienced in this field fully understand now that reinforced concrete, properly designed, does not fail all at once, as it was originally expected to, but deflects gradually, like a wooden beam, and shows plain evidences of overload long before failure actually occurs.

Turning now for a moment to other buildings with reinforced concrete floors in the San Francisco fire: The Academy of Science Annex, built in 1887, had floors of steel girders spaced 12 ft. apart and the space between filled with a flat slab of stone concrete 9 ins. thick reinforced with ¾-in. Ransome square twisted steel bars spaced 12 ins. apart. These concrete floors, according to Mr. Himmelwright, are generally in good condition. In the basement a reinforced concrete beam had its rod reinforcement exposed by the spalling off of the concrete below. Although the beam is badly cracked, it still supports the floor above. Mr. Himmelwright also shows an example in this building where the rods in the floor slab were exposed by the spalling off of the concrete below, yet the floor remained intact and does not appear to be seriously deflected. It is fair to say that the amount of combustible matter in the building was not great and the fire was not as intense here as in many other buildings.

Another building in which a severe fire occurred, the results of which are instructive, was the Young store and loft building. This had steel girders spaced 15 ft. apart, the space between them being filled with a floor of the expanded metal "suspender" type. The floor slab was of cinder concrete about 4 ins. thick with No. 16 expanded metal 3-in. mesh imbedded in it. Under the floor slab were concrete ribs spaced 4 ft. apart and supported on the under side by 5 × ⅜-in. flat steel bands or suspenders hooked over the top flanges of the girders and curving downward so as to make the rib 8 ins. deep at the center of the span. In this case it will be seen that the steel reinforcement of these concrete ribs was directly exposed to the fire, and must have been so expanded by the heat as to render little or no assistance to the concrete; yet the floors, for the most part, remain in place, although about half the area of the third and fourth floors sagged from 3 ins. to 12 ins. in the 15-ft. span. The first and second floors are in fair condition, and this is the case although, according to Mr. Himmelwright, the cinder concrete was of poor quality.

It seems certain that the strength of these floors was due not to the concrete ribs but to the strength of the flat floor slab itself. That this slab, only 4 ins. thick with a span of 15 ft., stood up without entire failure appears to us a remarkably good showing for reinforced concrete.

It is to be noted further that there were no cases whatever of reinforced concrete columns in the San Francisco buildings. There were a great number of cases, however, where steel columns and cast-iron columns failed, and Mr. Himmelwright himself remarks on p. 255: "All the ordinary and accepted methods of column protection were shown to be insufficient under the severest conditions." Concrete fireproofing, when properly anchored to the columns, however, he finds gave the best results. In view of this statement and of the astonishingly good showing made by concrete floors of short span under the severest conditions, it is at least open to question whether a reinforced concrete column properly designed would not offer greater resistance to fire than any type of steel column, protected by fireproofing.

We may here remark also that the numerous cases of the failure of steel columns by buckling and telescoping which Mr. Himmelwright shows suggest that the filling of steel columns with concrete, often advocated in this journal, would have greatly improved the fire-resisting qualities of the San Francisco buildings.

For partitions, reinforced concrete, according to Mr. Himmelwright, proved entirely satisfactory. While there were only a few examples of this construction found, "in every case they developed good fire resistance" and remained in much better condition after an hour or two exposure to fire than either the metal lath and plaster partitions or the partitions of hollow tile blocks. Mr. Himmelwright, therefore, expresses high approval of reinforced concrete for partitions and curtain walls.

Summing up now: the conclusions regarding reinforced concrete that we should draw after study of Mr. Himmelwright's investigation, are that this material on the whole has shown an astonishingly high resistance to fire. The lesson is impressed, surely, that the steel reinforcement should be protected from the heat by a good thickness of concrete, that cinder concrete is preferable to stone, that the flat slab construction for floors develops higher resistance to fire than a construction in which reinforced concrete beams and girders are exposed. Furthermore, where the highest resistance to fire is desired, as in warehouses where combustible goods are stored, either plenty of columns should be used, so that flat slab floors of moderate span may be used, or else steel girders thoroughly protected may be used in combination with flat slab floors. For ordinary buildings of office or residence type, however, almost any type of reinforced concrete is so vastly in advance of the ordinary wooden floor construction and even superior in fire resistance to the hitherto standard forms of steel and tile that it may well be accepted as satisfactory.

In the work of fire-resisting construction, it must be remembered that perfection is not to be had. Cost is always to be weighed in the balance against quality. Not one building in a hundred tested by fire will ever have to endure so fierce an ordeal as was experienced in the San Francisco conflagration; for under all ordinary circumstances water is thrown on a fire before the heat has time to penetrate far into the interior of the fireproofing materials used. Instead of condemning long span beams and gird-

ers of reinforced concrete, let us reflect that in the great majority of buildings now being erected for shop and mill and mercantile purposes, steel beams and girders and trusses are used wholly without protection from the action of flames. Look at the labyrinth of steelwork exposed to fire over nearly every machine shop and foundry and car barn and power station and a great variety of other industrial buildings. Is there any better plan in sight for reducing the fire risk in all this class of construction than the substitution of reinforced concrete girders and trusses?

When one looks at the question from this point of view, it is evident that the use of reinforced concrete has only just begun and that its advantages are only beginning to be appreciated. Mr. Himmelwright's valuable study of the San Francisco ruins will probably lead designers to take more care in imbedding their steelwork deeply in the concrete, and perhaps to the development of systems for better securing the concrete to the steel to prevent spalling off on exposure to heat. Cinder concrete of good quality instead of stone, also, ought to be more generally insisted on, wherever fire resistance is an important consideration. In bringing about these improvements in practice—or let us say the best practice, for there are unfortunately not a few working in this field who care for nothing but to erect the cheapest structure which will yield the largest profit—Mr. Himmelwright's book should render a valuable service to the engineering profession and the whole building industry.

LETTERS TO THE EDITOR.

Standard Specifications for Portland Cement.

Sir: I notice that in your issue for the present week, in your article entitled "Lowering the Tunnels Under the Chicago River," you state that "it may be noted that the cement used is required to comply with the specifications recommended by the American Society of Civil Engineers."

If this statement is correct, I am afraid the engineers in question have made a mistake, for if you write to Mr. Charles Warren Hunt, the Secretary of the Society, you will find that the American Society of Civil Engineers has never adopted any set of specifications for cement. The Committee on "Uniform Tests of Cement" made an extended report on what tests should be made upon cement and a very fine description of how each of these tests should be made, but it never recommended any set of specifications for the cement itself.

The American Society for Testing Materials took this recommendation up some time ago and adopted a set of standard specifications for cement, incorporating the recommendations of the above committee of the American Society of Civil Engineers and their own specifications in a pamphlet, of which I have a copy. A copy of this pamphlet may be obtained from Mr. Edgar Marburg, Secretary of the American Society for Testing Materials, University of Pennsylvania.

Yours very truly,
Walter M. Smith,
M. Am. Soc. C. E.

46 Elliott St., Dover, N. J., Sept. 15, 1906.

Comments on "Some Characteristics of Coal as Affecting Performance of Steam Boilers."

Sir: Referring to the extract of the paper, entitled "Some Characteristics of Coal as Affecting Performance of Steam Boilers," by W. L. Abbott, appearing in your issue of Sept. 13, I would call attention to the descriptive matter under Fig. 7 on page 277, which reads: "Value of Screenings as Compared with Pea Coal for Different Percentages of Ash and Different Sizes of Coal." It should have read: Coal Values for Screenings, because the values given were not made in comparison with pea coal. The theory upon which this table is based is, that 1¼-in. screenings containing 12% or less of ash and 31% or a less amount which will go through a screen having ¼-in. round holes is good enough for the purpose, therefore may be considered as of 100% value, but any screenings containing either more ash or more of the quantity which would go through the ¼-in. screen, or both, are of less value than 100% in proportion to the increase in ash and dust.

Referring to the title of the paper as presented, the exact reading was, as Affecting Performance *with* Steam Boilers, which is correct, as the coal can have no influence upon the boiler itself. If the coal changes in size or in quantity of ash, this cannot cause the boiler to act any differently, for the reason that its efficiency is fixed and permanent and cannot change unless its mechanical structure is altered; therefore the variation in coal does not affect the boiler, but it does affect the result produced through the boiler, or, in other words, the efficiency and capacity of the whole apparatus on which the different fuels act as variables and the boilers as a constant. This matter has been treated in detail in the Transactions of the American Society of Mechanical Engineers, Vol. 26, pages 418 and 619.

Very truly yours, A. Bement.

American Trust Bldg., Chicago, Ill., Sept. 14, 1906.

Cross-Section Books or Separate Sheets for Taking Topography.

Sir: In your issue of Sept. 6 I note the letter of Mr. F. Lavis comparing the practice of taking topography in cross-section books and transferring to the working map against the method of taking it upon separate sheets. Having had the opportunity to compare these methods in the matter of cost, speed, and accuracy in actual practice, I would like to submit the results of the work.

The methods used were the old style cross-section book method, transferring to a large working map by the field draftsman, and by making a plot of the alinement notes of the preliminary upon sheets of detail paper 18 ins. wide and in rolls of from one to five miles in length on the same scale as the working map. These sheets were placed on a home-made roll or plane table which was carried suspended from the topographer's neck like a peddler's tray. A straight-edge scale and hand compass were also carried.

Two rodmen were required in both cases. There is no reason for a smaller number of rodmen in one case than in the other if the same speed is to be maintained. Two miles of line were taken, one mile by each method. The conditions were almost identical; rough foot-hill work along a river, with its steep hillsides and canons of tributaries. Contours of 25 ft. were taken, and the limit for distance out was 1,200 ft. when ten contours would not be reached inside of that distance. The cost of the work compared as follows:

Cross-section book method:	
Topographer, 1 day	$3.00
Two rodmen, 1 day	4.00
Field draftsman transferring notes from cross-section book to map, 5 hours	2.00
Total	$9.00
The field sheet and transfer to plan by carbon paper method:	
Topographer, 1 day	$3.00
Two rodmen, 1 day	4.00
Field draftsman transferring to plan, ½ hour	0.20
Total	$7.20

Besides, the following should be considered as a saving in the cost of the work and a measure of the comparative accuracy. One mile of definite location, run from a projection made on the working plan, required fifteen changes on the ground. This means the slight shifting of curve points to get the line on the ground intended, and was necessary on the work done by the cross-section book method. Only five changes were required on the mile on which the topography was taken on the sheets. This caused the work done on the book mile to require 2 days while the sheet mile required only 1¼ days, effecting a saving of cost of $18 and making a total saving of $19.80 for one mile of work.

Another comparison of the accuracy attained by the method was afforded by the results of estimates made on the profile of projected locations as compared with the same on the profiles of the definite locations. The projection made on plan taken by the sheet method one mile, was off 1 yd. in 100 and the projection made on the plan from the book method was off 1 yd. in 40.

The results of the comparative work were so conclusive that I abandoned the book method altogether as being inferior in every respect to the sheet method. The matter of rain spoiling the sheets is not to be considered, as rain will ruin a cross-section book worse than a sheet, the lines being crowded on the smaller scale, and it makes no difference with the sheet as it is transferred to the map and kept only for record or thrown away.

If Mr. Lavis would try the sheet and transfer method, he would never want to use a cross-section book again.

As Mr. Lavis states, I have found it more economical in the use of time in making locations to use more care in running preliminaries and running none where it would not be possible to run a practical location within 100 ft., on the ruling conditions required. Another conclusion was that it saves time and obtains the very best results to always run a preliminary survey, take topography, and then project the definite location before staking it out. Wild fly-lines run at the rate of ten miles per day may create an impression of a high grade of skill on the part of the locating engineer with some people, but a close inspection of the work will reveal countless flaws in the detail.

Very truly yours,
J. H. Eby, E. M.
(Formerly Locating Engineer, A. & C. R. R.)

Durango, Colo., Sept. 11, 1906.

Plate Steel, Factors of Safety, and Commercial Testing.

Sir: I was very much interested in the way you handled my article on "The Segregation of Steel," presented before the American Society for Testing Materials at the Atlantic City meeting, in your issue of July 26, 1906, but I think you have drawn exactly the opposite conclusion as to the results in actual practice which should follow this exposition. You state at the close of the first paragraph that "our knowledge of the strength of the material remains restricted and uncertain, so that *high factors of safety* must be used." Now, the whole purpose of my article was centered about the present practice of using high factors of safety in certain forms of construction, notably the marine boilers which are inspected by the United States Board of Supervising Inspectors, this practice resulting in the use of hard and brittle steel in thick boiler plates, and thereby introducing a dangerous element, as I explained in my article.

Now, I believe that the facts set forth in my article would warrant exactly the reverse: the use of a lower factor of safety, because the tests are made on the softest part of the plates as a rule (in the conditions which necessitate the selection of the test pieces from that outer portion of the material); consequently all the center of the material is harder and higher and averages higher than the tests generally obtained. This would warrant the use of an average taken of all the tests or an assumption of an average higher tensile strength than that shown in the tests, and thereby warranting the lower factor of safety. In fact, I have practically come to the conclusion, because of the facts set forth in my paper, that practically all material that is called mild steel, ranging, say, from 50,000 to 70,000 tensile strength under average conditions, might be taken on one basis and used on one unit strain, say, 12,000 lbs. per sq. in., for structural work, and higher or lower for other classes of work, as the situation might warrant, thus getting a more satisfactory practice and doing away with a great deal of friction between inspectors and manufacturers of material.

All of this material between these ranges of strength, under standard conditions, would be equally trustworthy for this unit strain, unless it should be deficient in soundness and proportionate ductility, because, as I said in my remarks at the latter part of the article, the softer range has, other things being equal, greater ductility, and on that account would be safe to use up to a larger proportion of its ultimate strength, and results in service demonstrate very clearly that it is not the sagging down from soft material, but the cracking of brittle material (or else parts absolutely lacking in sufficient size) that cause danger. The present practice of allowing a strain proportioned to the actual tensile strength obtained from the piece introduces a commercial temptation to use high steels, causing the double error of light section and high tensile strength material, as a heavier section of softer material would cost more.

Yours very truly,
Charles L. Huston,
Vice-President Lukens Iron and Steel Co.

Coatesville, Pa., Sept. 10, 1906.

[That a high factor of safety necessarily calls for high-tensile-strength material, as Mr. Huston seems to say, we do not believe. In fact, the strength of the material as such has absolutely no relation to the proper factor of safety, provided the strength is fairly constant and is determinable with reasonable accuracy. The considerations that fix the margin of stress to be allowed, or in other words fix the factor of safety, are of different kind. Our view of this subject was somewhat fully set forth a few weeks ago, in an editorial article in our issue of Sept. 6, 1906, P. 258.

It is probable that Mr. Huston refers to that state of affairs where the working unit stress has been adopted at the start, without direct reference to commercial material, and later the endeavor is made to protect this stress with as high a factor of safety as possible, by purchasing high-strength material. Such practice, or the attitude which leads to such practice, is of course distinctly reprehensible. Mr. Huston's quarrel is with this practice, we take it, and not with high factors of safety in the abstract.

As to the rest, we are in general agreement with the position taken in the above letter. It is not desirable or proper to push tests on the softest parts of plate steel up to the specification requirements, without regard to the condition of the middle part of the plate. Or, to be more exact, it is improper to frame specifications in such a way as to require tests of an *abnormal* portion of the material. In some few cases it may be commercially impossible to test the normal part of the piece, but in those cases the

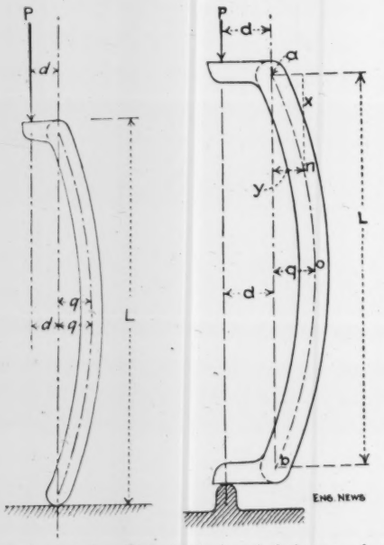
specification should recognize this fact, if possible, fix a factor expressing the usual relation or normal to abnormal, and base its stresses and its requirements on the normal strength. In plate and structural shape steel this is a particularly important matter. As a preliminary step toward the realization of these ends it is highly desirable to get on record full data on all variations of the kind referred to, in commercial materials.—Ed.]

On the Analysis of Columns.

Sir: Since discussion is invited on Mr. Cardullo's theory of eccentrically loaded columns, presented on p. 262 of your issue of Sept. 6, the present writer would offer the following.

Mr. Cardullo would seem to have in view the establishment of formulas more simple, and better adapted to practical use, than the transcendental equations resulting from the rigorous mathematical treatment to be found in some of the text-books (those of Profs. Merriman and Bovey, for example) for the case of columns under eccentric loads. In his procedure he has virtually assumed the character of the elastic curve, instead of leaving it to be determined by the conditions of the problem; and in this respect has followed the same mode of treatment as that presented in §269 of Vol. I. of Weisbach's Mechanics (see p. 544 of Coxe's translation) where the form of curve is taken as the arc of a circle. The results of the two suppositions differ only in the value of a single numerical constant.

By inadvertence, doubtless, Mr. Cardullo's figure (here reproduced as Fig. 1) presents a very anomalous situation. No possible value or direction of the reaction at the lower extremity of the column in this figure could equilibrate the vertical force P in the position shown, and if the column were tilted so that the vertical line



Figs. 1 and 2. The Analysis of Columns.—I. P. Church.

of action of P could pass through the point of support at the lower end, the moment of P about the middle point of the elastic curve would be P (½ d + q) and not P (d + q). However, what was probably intended is the situation shown in Fig. 2, where the reaction at the lower extremity has the same eccentricity, and is on the same side, as P itself. (Or, the upper half of the present column could be regarded as independent, with its lower end "built in," i. e., with a vertical tangent at the point o; and this would present the same problem.)

Here we have an elastic curve, a o b, and the deflection q is a definite function of P and d (as we know from the rigorous treatment mentioned above) and becomes fixed in value if P and d are given. Now Mr. Cardullo computes the value a vertical force W should have if, acting at a, its moment about point o is to be equal to that of P about o; i. e., $W = P (d + q) \div q$, and then assumes that this force W must be equal to the critical value given by Euler's equation for the load producing "incipient flexure" when applied without eccentricity to a slender column of the same dimensions as the one under treatment. But this critical value is independent of the deflection q; which seems to indicate that Mr. Cardullo's assumption is wide of the truth. The fallacy would appear to reside in the statement:

"We know that the moment of the eccentric force which maintains a given deflection q must be the same as the moment of the load W. Therefore, etc."

This is equivalent to claiming that the deflection of a point in an elastic curve depends only on the bending moment at that one point, whereas such deflection is actually dependent on the bending moments (and consequent differing degrees of curvature) at all the points along the curve.

For instance, let us suppose that P were removed and that immediately this specially computed load $W = P (d + q) \div q$ were placed at the point a; the point of support underneath being simultaneously changed to b. Would the elastic curve change form (and consequently q change its value)? It most certainly would; for the reason that, although the bending moment at o remains unchanged just after the replacement, the moments at all other points along the column, such as n, would just at first be less than their original values, and an elastic movement would surely occur. In fact, the column would straighten up, thus showing the above value W to be less than Euler's critical load.

It would seem, therefore, that, from the point of view of strict theory, Mr. Cardullo's treatment of the problem is very faulty. Nevertheless, his results may be shown to be rudely approximate by comparing them with the result of the approximate method referred to above in Weisbach's Mechanics.

If the bending moments along the axis of the column were all equal, the elastic curve would be a circle. Evidently this situation would be the more approximated to, the greater the value of d compared with q. With rude approximation, then, the elastic curve will now be assumed to be part of a circle (see Fig. 2) whose radius may be called R; in which case we have from simple geometry

$$R = \frac{L^2}{8q} \quad \ldots\ldots\ldots\ldots (1)$$

while from the common theory of flexure

$$P (d + q) = \frac{f_1 I}{e} \quad \ldots\ldots\ldots\ldots (2)$$

and

$$P (d + q) = \frac{E I}{R} \quad \ldots\ldots\ldots\ldots (3)$$

whence also

$$R = \frac{E e}{f_1} \quad \ldots\ldots\ldots\ldots (4)$$

Eliminating between these four equations we find

$$f_1 = \frac{8 P d . e E}{8 E I - P L^2} \quad \ldots\ldots\ldots\ldots (5)$$

on adding which to the direct compressive stress

$$f_2 = \frac{P}{A} \quad \ldots\ldots\ldots\ldots (6)$$

there is finally obtained

$$f = \frac{P}{A}\left[1 + \frac{8 e d . E A}{8 E I - P L^2}\right], \quad \ldots\ldots\ldots\ldots (7)$$

as the total compressive stress in the outer fiber on concave side of column at middle section o (for the mode of support, etc., of Fig. 2).

It seems to the writer that the best way to present Mr. Cardullo's treatment would be somewhat as follows:

Let us assume as a rude approximation that the curve o a b or elastic curve of the column, is of the same form as if the column were subjected to an axial load at a (and reaction at b); then by Euler's relation the bending moment at o would be $\left(\frac{\pi^2 E I}{L^2}\right) q$ and we may therefore write $\frac{E I}{R} = \left(\frac{\pi^2 E I}{L^2}\right) q$, (where R is the radius of curvature of the curve at the point o) whence it follows that

$$R = \frac{L^2}{\pi^2 q}, \text{ which (say) } = \frac{L^2}{10 q} \quad \ldots\ldots\ldots\ldots (1a)$$

This relation will now take the place of eq. (1) and when combined with (2), (3), (4) and (6) leads finally to Mr. Cardullo's result for a pin-ended column eccentrically loaded, viz.:

$$f = \frac{P}{A}\left[1 + \frac{10 e d . E A}{10 E I - P L^2}\right] \quad \ldots\ldots\ldots\ldots (7a)$$

It is seen that this does not differ greatly, as such things go, from eq. (7), based on the assumption of a circular arc.

Weisbach follows up his treatment in the case of iron, and finally derives for ordinary modes of loading, the form

$$P = \frac{C' A}{1.45 + B'\left(\frac{L}{D}\right)^2} \quad \ldots\ldots\ldots\ldots (8)$$

for the breaking load of a solid cylindrical column of diameter D; whereas the Rankine form is

$$P = \frac{C A}{1 + B\left(\frac{L}{D}\right)^2} \quad \ldots\ldots\ldots\ldots (8a)$$

C, B, C', and B', being constants for the material.

I. P. Church.

Ithaca, N. Y., Sept. 9, 1906.

Sir: In your issue of Sept. 6 Mr. Forrest E. Cardullo develops what he considers a set of new formulas for the design of columns, and from these formulas he makes a number of deductions to guide one in practical designing. These deductions are so contrary to accepted practice that Mr. Cardullo himself is diffident about the proposed theory.

The formula

$$f = \frac{P}{A}\left[1 + \frac{K e d E A}{K E I - P L^2}\right]$$

deduced in that article is identical with the well-known standard formula for the compressive unit stress of an eccentrically loaded column,

$$f = \frac{P}{A} + \frac{M e}{I - \frac{P L^2}{K E}}.$$

The identity may be verified by substituting P d for M in the standard formula. This formula can *not* be applied to a centrally loaded column because the initial eccentricity is wholly unknown. If the initial eccentricity is "indefinitely small," then the lateral flexure is just as "indefinitely small" and that does not help the designer any. The denominator, $K E I - P L^2$, never approaches zero in practical work because that would generally give too slender a member; thus in order to make $P = K E I/L^2$ a 4 × 4-in. pine stick 19 ft. long, with square ends, would safely support 16,000 lbs. and an ultimate load of about 100,000 lbs.—as absurd a result as one can get by interpreting formulas without practical consideration.

The conclusions drawn by that writer are all based on the false assumption that the two distinctly separate causes for the failure of the column, lateral deflection and direct pressure, act distinctly separate. The reasoning in the design of a centrally loaded column is this: We will *try* to load the column through its center of gravity, we know that it is impossible to do this exactly, the least eccentricity will tend to develop greater and greater flexure. This flexure of the column causes compression in the fibers of the inner side of the curve, and tension on the outer side. This compression will combine with the direct compression in breaking the beam. Not knowing how much eccentricity there is, the only safe provision in designing the column is to assume that the load is sure to deflect it, like a rattan cane. The effect of the deflection is proportional to $(r \div l)^2$ according to Rankine's theory or proportional to $r \div l$ according to the straight line theory. The constants are in either case chosen to suit the results of tests within practical limits—those limits being much lower than indicated by Euler's theory.

Taking up Mr. Cardullo's deductions one by one.

1. "The greatest stress produced in any column by a load less than the elastic strength," with an initial "indefinitely small eccentricity" will be greater than "the load divided by the sectional area" because of the flexure moment W q; the deflection q depends on the value of $r \div l$, and its effect for an ordinary range of values is to weaken the column by from 5 to 50%.

2. "If the load exceeds the value $K E I \div L^2$," the column will fail from the *combined* effect of direct compression and flexure.

3. "If the load does exceed the elastic limit" it will probably fail as in case 2, before it reaches that point.

4. "If I be 'sufficiently' large" the column will fail by crushing; to be sure, if the column is a cube it will. But for ordinary values of I, L and $r \div l$ the effects of lateral deflection and direct compression will appear together.

The consequent suggestions to design columns first for inflexibility making $r^2 = F L^2/E K$, then for compressibility making $A = P/f$, are based on the assumption that the two stresses are distinct and act separately on a column; when, as a matter of fact, the column is an unconscious body, and does not analyze the source of the stresses it is subjected to; its fibers are compressed by the direct load *at the same time* that they are compressed by the bending of the beam. The direct safe unit load must therefore be such that when it is added to the compression caused by the bending the sum will equal to the safe unit compression of short members. Rankine's formula as well as the straight line formulas provide for this, Mr. Cardullo's interpretation of the formula does not.

Sincerely, Louis Ross,
Assistant Engineer, U. S. Reclamation Service.
Washington, D. C., Sept. 13, 1906.

A Knotty Problem in Stress-Analysis; Dangerous "Safe Stresses" in a Reinforced Concrete Bridge.

Sir: I enclose herewith sketches of a small reinforced-concrete bridge which, according to my estimates, is subjected to an intensity of stress far beyond safe limits. I have referred the matter of the strength of the design to the engineering firm which submitted the plans and specifications to the County Commissioners, and it assures me that the design is safe and has been checked. I am unable, however, to secure from them any proof that the design is safe, in

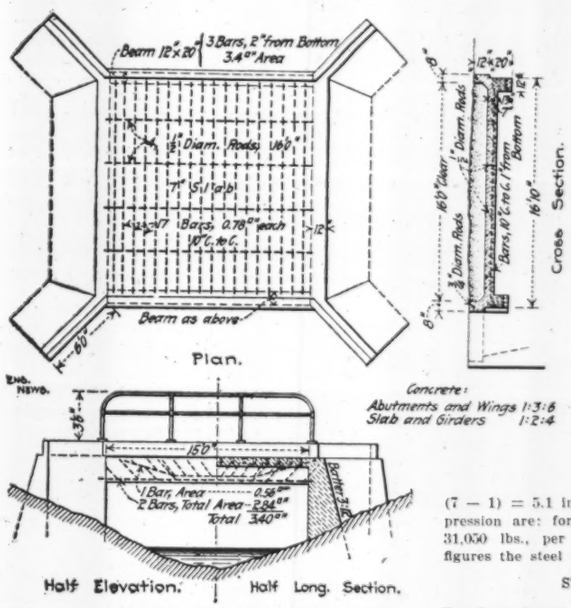


FIG. 1. A REINFORCED-CONCRETE GIRDER BRIDGE WHICH HAS JUST BEEN BUILT IN INDIANA.

the way of method of analysis. I am curious to know whether my analysis is in error, and would like criticisms of my results from your readers.

The sketch (Fig. 1 herewith) shows the bridge according to the working drawings by which it was built. These differ from the filed plans in being somewhat more complete. The plans provide for a 7-in. concrete slab, spanning 16 ft. between longitudinal girders, which latter have a span of 15 ft. between abutments; the slab is reinforced with bars whose cross-section area is $^{11}/_{16}$-sq. in. each, spaced 10 ins. apart. The centers of these bars are to be 1 in. above bottom of slab. The slab is to be covered with 12 ins. of earth fill.

The girders are 12 ins. thick and project 13 ins. below the slab, and are fully shown as to dimensions and reinforcement on the drawing, but I have not estimated the girders and my remarks apply only to the floor slab. The top of the girders was a coping merely, as shown in the sketch. In this respect the working drawing conforms to the filed plan; but the contractor modified this in building by substituting concrete railings 8 ins. thick and omitting the coping.

The filed plans contained substantially the following load specification:

Bridge designed to carry 15-ton road roller or a uniformly distributed load of 200 lbs. per sq. ft.

This is not entirely in accord with the printed specifications, but is practically so; the discrepancy is probably due to haste in preparing plans. It is generally well known that the county in question insists on having all bridges designed for 200 lbs. per sq. ft. of uniform live load or 20-ton road roller.

The specifications, drawn by the engineering firm which submitted the plans, provide:

Maximum compression in the concrete, 400 lbs. per sq. in.
Maximum tension in the steel, 16,000 lbs. per sq. in.
These bridges [some 10 or 12 similar bridges are involved in the same or parallel contracts—D. B. L.] are to be built . . . and in strict accordance to the plans. Arch* and abutments should be made sufficiently strong to carry their own weight, weight of filling, road, spandrel walls, and the superimposed load as follows:
Road Roller, 20 tons.
2 rear wheels each 12,000 lbs.
2 front wheels together 16,000 lbs.
Right is reserved to test any portion of the above structure by applying twice the calculated load. If in consequence any ill effects occur, the damage must be made good and all work must be strengthened so as to meet the requirements of these specifications without expense to the owner.

According to my analysis of the stresses in the slab, I estimate that, for dead load alone, the compression in the concrete is about 1,000 lbs. per sq. in., and the tension in the steel nearly 20,000 lbs. per sq. in. For the live load as specified (20-ton roller), I find the compression in the concrete to be 3,000 lbs. per sq. in., and the tension in the steel 55,000 lbs. per sq. in. But according to the specifications the bridge is to be tested under twice the specified live load!

It is perhaps unnecessary to observe that the indeterminate distribution of the concentrated loading of the road roller makes the analysis of the slab subject to discussion. No wheel or axle spacing is given. In my calculations I have assumed the rear wheels of the roller to give maximum loading and have spaced them 5 ft. apart. Further, I have assumed their weight to be distributed over a strip of floor slab 5 ft. wide (in the direction of the bridge axis).

The dead load, taken at 80 lbs. per sq. ft. for the 7-in. thickness of concrete and 120 lbs. per sq. ft. for the 12 ins. of earth, totals 200 lbs. per sq. ft. This gives a dead-load moment at center of slab, of 76,800 in.-lbs. per foot of width. The two 12,000-lb. rear wheels, with one wheel at center of roadway or with the wheels placed symmetrically to center, give a center moment of 158,400 in.-lbs. per foot width. In this case, as above remarked, I assume the axle load to be carried by a strip of the slab 5 ft. wide. Taking the effective depth of the beam as $0.85 \times (7 - 1) = 5.1$ ins., the values of total tension or compression are: for dead load 15,060 lbs.; for rear axle 31,050 lbs., per foot of width of slab. From these figures the steel stresses are obtained directly, and are:

STRESSES IN STEEL.

	Lbs. per sq. in.
For Dead Load	18,250
Dead Load plus Rear Axle	55,900
Dead Load plus twice Rear Axle (test load)	93,550

The stress in the extreme fiber of the concrete may be approximated by taking 2½ ins. as depth from top of slab to neutral axis, which, with rectilinear stress distribution, gives the following:

STRESSES IN EXTREME COMPRESSION FIBER.

	Lbs. per sq. in.
For Dead Load	1,000
Dead Load plus Rear Axle	3,070
Dead Load plus twice Rear Axle (test load)	5,140

The bridge has been erected according to these plans, as well as numerous other similar bridges, and has not yet deve'oped defects, so far as I know; but this of course does not prove that it is safe. None of the bridges have been subjected to tests as specified, nor to maximum live load. I am curious to know whether this design would be considered a safe one.

Yours truly, Daniel B. Luten.

Traction Bldg., Indianapolis, Ind., Aug. 11, 1906.

[After this letter was in type, and while preparing to send proof to the author for correction, we received the following further letter, which shows that the doubts above expressed as to the strength of the bridge found startling confirmation.—Ed.]

Sir: On Aug. 11 I mailed you sketches and results of my analysis of a concrete bridge which had just been contracted for [see preceding letter.—Ed.]. Several similar bridges were then building or had been completed in the same region. On Aug. 18 the very bridge referred to collapsed under a traction engine the day following removal of centers. I visited the location on Aug. 20, and secured the accompanying photograph [Fig. 2]. The engine had been removed from the stream on the day of the accident, and a temporary wooden bridge erected.

In an interview with the engineer, I learned that the engine was a 14-HP. machine, built by Avery Mfg. Co., Peoria, Ill., weighing approximately 16,500 lbs. with full boiler, of which about 9,000 lbs. was concentrated on the rear wheels. The engineer stated that the engine had advanced across the bridge until the front wheels were nearly on the farther abutment and the rear wheels about at the middle of the 15-ft. span, when the structure suddenly gave way. He was unable to say what part of the bridge gave way first, but the indications are that the floor slab failed at the middle of the span, precipitating the entire structure into the stream at the same time. The engineer was imprisoned between the footboard which was bent upwards, and the canopy, but escaped without injury. The separator was coupled behind the engine, but the coupling broke and it stopped within a foot of the opening. Had it followed the engine into the wreck, it would doubtless have crushed the engineer.

A careful examination disclosed no broken steel, the failure apparently having been caused by excessive compression in the concrete, as was to have been expected if my analysis of stresses, as given in my previous letter, is correct. The concrete appeared to be of good quality, and although badly fractured and broken, large pieces would ring true when struck with a hammer.

Besides this bridge and another of exactly the same dimensions there have been completed in the same county five others of similar design but different spans. I examined one of these, 24 ft. in span, and found the middle concrete beam badly cracked. The collapsed bridge and this bridge were completed early in July, and the centers removed about the middle of August. The contractors had rendered bills to the county at the July session for these two bridges, but for some reason the Commissioners had not allowed the bills, possibly feeling that the bridges should first be tested as provided in the specifications. None of the seven bridges are paid for, and it is an open question as to who shall assume the responsibility for the wrecked bridge, and what is to be done with the other bridges which are also possibly unsafe. According to my analysis the responsibility should be on the designers, but the contractor seems to have assumed it by accepting the contract on the specifications and plans. It is my belief that the designer

FIG. 2. PHOTOGRAPH OF REINFORCED CONCRETE BRIDGE WHICH FAILED UNDER 16,500-LB. TRACTION ENGINE TWELVE HOURS AFTER BEING THROWN OPEN TO TRAFFIC.

Span 15 ft. Width 16 ft. The timber temporary bridge is on the site of the collapsed structure. The walls in foreground are the up-stream wings. This is the same bridge shown in Fig. 1.

*I do not know why the word "arch" was used, except that the specifications were a printed copy attached to several drawings, some of which were arches.—D.B.L.

had been accustomed to floor loads in buildings, and had failed to take into account the weight of the earth fill on the bridge, or to allow for the heavy concentration of wheel loads due to a road roller.

The engine was damaged to the extent of possibly $100, including the cost of rescuing it from the bed of the stream. The contract price of the bridge which collapsed was $300. The abutments and wings are still serviceable, but the girders and floors and the reinforcing bars are a total loss.

Very truly yours,

Daniel B. Luten.

Indianapolis, Ind., Aug. 24, 1906.

[In order to give opportunity for an explanation of the matters above referred to, a proof of Mr. Luten's letter was submitted to the firm which from the information in hand appeared to have made the design and the calculation. The following Report and Calculations were received as a result.—Ed.]

REPORT ON FAILURE OF REINFORCED-CONCRETE BRIDGE.

On Saturday, Aug. 18, at 8 a. m., Garrison's traction engine hauling separator and passing eastward over this bridge went down with bridge when traction wheels struck the span. The proper ballast had been placed on the slab. Forms had been removed Wednesday, about three days before, after remaining in place three weeks from time concrete was placed.

Causes of failure were:

1. Poor concrete.
2. Bars not placed properly in beams.

No one hurt. Engine not damaged by accident. Threshing gang of 22 men got engine out in four (4) hours.

Remarks:

Concrete very poor—of several grades—and in no case where steel was exposed after accident could concrete be found adhering to steel. All such steel surfaces were almost as clean as when placed. In general, dirt was used instead of sand in the aggregate.

Beam bars, instead of being side by side 2 ins. from bottom, were at ends 4 ins. to 14 ins. from bottom of beam and one above the other.

The 7-in. floor slab with bars 10 ins. center to center was so covered with debris that none but general deductions can be made. It did not seem badly broken.

General carelessness in construction can be deduced from the bars exposed along the sheared face of the abutments, which in each case show floor bar on the abutment throughout the length of the bar. This first bar should have been 5 to 10 ins. from the face of abutment. Conclude from this that the floor slab proper had too few bars, that is, less than was called for in the design.

Floor bars did not seem to reach well into beam, and slab seemed to shear away along the side of the beam. Clear span was 14 ft. 10 ins. and 16-ft. bars would have 7-in. bearing at each end, which is ample.

On north side one bar to go in beam was not in beam at all, but was placed in concrete rail 1 ft. above beam. Drawing showed 12-in. parapet with gas pipe rail; instead a 10 × 36-in. rail was built, loading beam without adding to its strength.

Concrete in ballast wall was not laid continuously with floor slab and beams, as per drawings.

Wing walls were carried down on back face without the 3:12 batter shown on drawing.

Could not determine how abutments were carried down on back face. Possibly without batter. Poor concrete in abutments. Wings have better finish.

CALCULATIONS OF STRENGTH OF BEAMS AND SLABS AS PER DRAWINGS.

SLAB:

14 ft. 10 ins. clear span, say 15 ft. 0 ins.; 7-in. slab; bars 10 ins. c. to c.; 0.78 sq. ins. area each.

Bars at 0.78 sq. ins., 10 ins. c. to c. = 12/10 × 0.78 = 0.93 sq. in. per foot wide.

7 ins. — 1 in. = 6; × 12 ins. = 72 sq. ins.

0.93/72 = 1.29%.

$Y = 6 \times 0.83 = 5.1$ ins.

$$M = \frac{W L \times 12}{8} = a f y;$$

Hence

$$W = \frac{8 a f y}{12 L} = \frac{8 \times 0.78 \times 16{,}000 \times 5.1}{12 \times 15} = 2{,}829$$

lbs. per 10 ins. wide.

12/10 × 2,829 = 3,396 lbs. per foot wide.

$\frac{3396}{15} = 226.4$ lbs. per sq. ft.

7-in. slab weighs 80 lbs. per sq. ft.

12-in. fill weighs 100

180 ÷ 2 = 90 lbs. factor of 2.

226 — 90 = 136 lbs. unit live load per square foot.

BEAM:

12 × 20-in. beam plus 2 ins. for ballast wall = 12 × 22 ins. = 264 sq. ins.

2 bars at 1.42 = 2.84 sq. ins.
1 bar at 0.56 = 0.56 sq. ins.

3.40 sq. ins.

$\frac{3.4}{264} = .012878$, or 1.28% reinforcement.

$Y = 20 \times 0.85 = 17$ ins.

$$W = \frac{8 a f y}{12 L} = \frac{8 \times 3.4 \times 16{,}000 \times 17}{12 \times 16} = 38{,}533$$

lbs. total.

D. L. Slab, 8½ ft. × 16 ft. × 80 lbs. = 10,880 lbs.
Fill, 8½ × 16 × 100 lbs. = 13,600 lbs.
Beam, 1 ft. × 2 × 16 × 150 = 4,800 lbs.

29,280 lbs. factor of 2

$\frac{29{,}280}{2} = 14{,}640$ factor of 4.

38,533 — 14,640 = 23,893.

$\frac{23{,}893}{8\frac{1}{2} \times 16} = 175.68$ lbs. live load which beam is good for per sq. ft. of floor.

Weight of engine loaded = approx. 16,500 lbs., or 9,000 lbs. on rear axle.

NOTE: Round rods in floor slabs distribute the concentrated loads on slab and with the fill above care for impact and concentration and make uniform distribution of loads on slab and beams.

[Without attempting full discussion of the above figures, we may note the fact that the concrete stresses are not given. But, if the steel carries 16,000 lbs. per sq. in., the concrete stresses are absolutely determined provided a ratio for E_s/E_c is assumed, and provided the stress of the concrete be taken to vary in a given proportion to the distance from the neutral axis. With linear stress distribution in the compression concrete, and no tension resisted by concrete, we have

$$f_c = \frac{f_s r}{100}\left[1 + \sqrt{1 + \frac{200}{rn}}\right]$$

which becomes, for $n = E_s/E_c = 15$,

$$f_c = \frac{f_s r}{100}\left[1 + \sqrt{1 + 13.3/r}\right]$$

In the present case r = 1.29 according to the calculation of the designer; hence the concrete compressive stress corresponding to 16,000 lbs. per sq. in. in the steel is

$$f_c = \frac{16000 \times 1.29}{100}\left[1 + \sqrt{1 + \frac{13.3}{1.29}}\right]$$

$$= 160 \times 1.29 \times 4.36 = 900 \text{ lbs. per sq in.}$$

This is a higher stress than is allowed by most specifications, whether those of municipal building bureaus or those of private parties, yet with good concrete it is not necessarily a dangerous stress. It is important to note that the above stress corresponds *not to the roller load*, but to an artificial loading composed of

First: *Half* the dead load.

Second: A uniform live load of 136 lbs. per sq. ft. in the case of the slab calculation, or 176 lbs. in the case of the girder calculation.

The subject of high unit compressive stresses in concrete in beams is discussed in the following letter, which we have received in connection with the preceding letters.—Ed.]

REGARDING AN ALLOWABLE STRESS IN COMPRESSION IN THE EXTREME FIBER OF REINFORCED CONCRETE SLABS AND BEAMS.

Actual full-sized testing is the one method to obtain results which may be used in practice in order to determine the allowable compression stress in the extreme fiber.

Of course a very large number of tests have been made, which agree very likely in the amount of compression which has been developed. A good example would be the tests prepared by Professor Talbot. In the two reports, issued by the University of Illinois, there is a statement made that in a test to destruction the beam would fail either by shear or bending before the concrete began to compress, provided not more than 1½% of steel is used.

Prof. Talbot's tests indicate that one would be justified in designing beams and slabs to contain in the medium steel 1¼%. However, this percentage of steel gives in the extreme fiber a compression of about 1,000 lbs. per sq. in., which is considered by many engineers as excessive. However, this 1-in. never acts alone. It is only a part of the compression flange, and the other portions, not being obliged to sustain so great a stress, naturally strengthen the extreme fiber.

The average stress in the compression flange will be two-thirds of the maximum, which would be approximately 666 lbs., which, for good concrete, is not excessive. At any rate, the writer has used 1½% in slabs and 1¼% in isolated beams on a very large amount of reinforced-concrete work. This has been tested and proved satisfactory and therefore the writer feels that the use of such a percentage of steel is justifiable.

In reinforced concrete work only the best concrete should be used and therefore all calculations are based upon such concrete. If there is the slightest tendency for poor concrete to be used or the slightest indication that such will be obtained through lack of skill or improper superintendence, then an extra factor of safety should be allowed.

Assoc. M. Am. Soc. C. E.

[We submit this entire case to the earnest consideration of engineers interested in reinforced-concrete construction.—Ed.]

THE NODULIZING AND DESULPHURIZATION OF FINE IRON ORES AND PYRITES CINDER.*

By Albert Ladd Colby.†

Those who have had practical experience in smelting large percentages of fine material in blast furnaces, and also in attempting, under these conditions, continuously to produce large outputs of uniformly high-grade iron, as well as those who have listened to explanations as to why the daily blast furnace reports do not show that the desired results are being obtained, appreciate, without explanation, the difficulties attending the smelting of an ore mixture containing considerable proportions of fine ore or other ferruginous materials, as well as the annoyance and expense of handling the large quantities of flue dust produced.

By consulting Patent Office files, more particularly of Great Britain, the United States and Germany, some idea can be formed of the ingenuity which has been brought to bear on this problem, and the many efforts which have been made, especially during the last ten years, in endeavoring more economically and successfully to utilize fine iron ores, flue dusts and ferruginous by-products. Under these conditions a process which has been in operation continuously, commencing last summer, and has successfully and economically converted over 24,000 tons of fine ferruginous ores and by-products into "nodules" which have been smelted in blast furnaces, without the difficulties accompanying the use of large proportions of fine materials, and has, furthermore, economically reduced the sulphur present in objectionable quantities in the fine material, is certainly worthy of attention.

The nodulizing plants using this patented process will be first referred to, as in this manner the scope of the application of the process can best be indicated, as also the fact that full advantage of the value of the invention has been obtained by the economic devices used in the various operations. This description of the plants will be followed by a short reference to the patented process.

GENERAL DESCRIPTION OF PLANTS.

Plant at Newark, N. J.—This plant was located with special reference to the delivery of pyrites cinder from various sulphuric acid works, at a low cost of freight, and also at a point convenient for the delivery, by rail, of the product to the blast furnaces of New Jersey and eastern Pennsylvania. It is situated on a 12½-acre tract of land on the meadows, at the junction of the Hackensack and Passaic Rivers, which unite forming Newark Bay, while on the north side is the Morris Canal, so that the tract is entirely surrounded by water. Beside dockage facilities, the plant is connected by rail with the Central Railroad of New Jersey, and, indirectly, with the Pennsylvania Railroad.

After a practical demonstration had proved that the patented process could be worked continuously and economically, and after the transfer of the patents by the inventor to the National Metallurgic Co., of New Jersey, the above tract of land was purchased in March, 1905, and the plant was built. It consisted of one rotary kiln, of a smaller diameter than has since been adopted as a standard in their more recent plants, which has produced an average of 75 to 90 tons of nodulized desulphurized pyrites cinder per day of 24 hours, the time occupied in the experimental nodulizing of other fine materials having greatly interfered with the output. The total amount of nodules shipped to blast furnaces from the Newark plant up to May 1, 1906, has been 9,477 gross tons.

The Newfoundland Syndicate, of which the National Metallurgic Co. is one of the constituent companies, own immense pyrites and copper pyrites deposits in Newfoundland. Their holdings include an area of about 125 sq.

*Condensed from a paper read before the Iron and Steel Institute.

†International Nickel Co., 43 Exchange Pl., New York City.

miles. Four large deposits are now being actively operated, delivering about 70,000 tons of the pyrites and copper pyrites roasted in the United States per year, which in 1905 amounted to about 600,000 gross tons. They also control, for the next five years, 80% of the cinder which will be made in the United States from the roasting of pyrites or copper-bearing pyrites. During the development of their mines now in operation, as well as by the result of surveys of their undeveloped properties, it became evident that the pyrites, as shipped, would contain copper in paying quantities. The addition to the Newark plant of a modernized Henderson copper-leaching plant was therefore at once decided upon and is now in operation.

Two other kilns have also been installed, so that the plant now consists of three kilns: No. 1, for nodulizing; No. 2, arranged by chimney and flue connections for either nodulizing or roasting with salt; and No. 3, intended entirely for the Henderson roasting. The smaller kiln, No. 1, and the standard kiln, No. 2, have together produced continuously some 250 gross tons of nodulized pyrites cinder per day of 24 hours. By next autumn this plant will be further enlarged by the addition of two more nodulizing kilns, which are now ready for delivery.

Plant at Aspinwall, near Pittsburg, Pa.—The plant located at Pittsburg contains two large nodulizing kilns with an estimated output of 250 tons of nodulized material in 24 hours. The kilns will be fired with natural gas. The plant has been located with regard to the nodulizing of the large quantities of flue-dust necessarily made by the many blast furnaces of the Pittsburg district, all of which use a large proportion of Mesaba and other fine Lake Superior hæmatites in their ore mixture, and also with regard to the convenient delivery of pyrites cinder from sulphuric acid works at Beaver Falls, Titusville, Cleveland, and other interior points.

Plant at Steelton, Pa.—A plant with one kiln was built and put in operation in June, 1905, by the Pennsylvania Steel Co. at their works at Steelton, Pa., operating under a license from the National Metallurgic Co. This plant has successfully nodulized magnetic concentrates, fine ores from the Mesaba district and the Lake Superior region, and flue dust from their blast furnaces mixed with the two former materials.

The Pennsylvania Co. are about to build, under their license, another plant of three nodulizing kilns for the treatment of fine ores; the nodulized product, estimated at 450 tons per day of 24 hours, will be consumed in their own blast furnaces.

The following analyses are representative of current practice in nodulizing fine magnetites at Steelton:

Constituents.	Magnetic concentrates.	Nodules from magnetic concentrates.
Iron	59.64	61.25
Sulphur	1.15	0.13
Phosphorus	0.01	0.01
Silica	8.50	8.61

Plant at Hazard, Pa.—The New Jersey Zinc Co. are owners of the celebrated mine of franklinite at Franklin, N. J. At two of their plants, situated at South Bethlehem and at Hazard, Pa., they crush this ore and roast it on open grates with fine coal, catching the zinc oxide in flues, and smelting the cinder left on the Wetherill grates in a small blast furnace with other ores for the manufacture of spiegeleisen. The fine franklinite, produced in the crushing prior to roasting, as it passed through an 80-mesh sieve, fell through the grates. It was therefore determined to try nodulizing this fine material into uniformly sized lumps. A trial lot of some 300 tons was very successfully nodulized at the Newark plant, with but little or no loss of zinc. The fine franklinite possessed the following chemical composition:

	Per cent.
Zinc	17.40
Manganese	8.50
Iron	15.76
Silica	11.37
Lime	16.14
Magnesia	1.44
Loss on ignition	11.18

The nodules of the desired size contained 20.4% of zinc. As the result of this experiment, a nodulizing plant has been erected at Hazard; it will be in operation very shortly.

This experiment shows that the kiln temperatures can be readily controlled, and that material containing volatile metals can be nodulized, practically without any loss of the valuable volatile constituents.

Plant at Oxford, Pa.—The Empire Steel & Iron Co., of Catasauqua, Pa., own and operate a large mine of magnetite, located at Oxford, N. J. The Oxford property covers bout 2,500 acres of iron-ore lands, and the company controls the mineral rights for several thousand additional acres. The vein is very well and sharply defined, and extends for at least two miles, at widths varying from 20 to 40 ft. The company is thus fortunate in owning what is probably the largest magnetite ore body in New Jersey. The shipments from one shaft now amount to 8,000 tons per month.

A representative analysis of the run of the mine is as follows:

	Per cent.
Iron	60.00
Sulphur	3.00
Phosphorus	0.48
Silica	6.00

There is a considerable quantity of ore somewhat lower in iron than the above, which is also very high in sulphur, as shown by the following analysis:

	Crushed raw ore. Per cent.
Iron	57.95
Sulphur	3.58
Phosphorus	0.597
Silica	8.72

With a view to ascertaining the best mode of dealing with the latter ore, the company obtained a trial run at the Newark plant of the National Metallurgic Co., of some 50 tons of crushed ore. The results of this trial were perfectly satisfactory as far as the nodulizing was concerned, but tests taken every hour during the 10-hour run, while showing low sulphur in some samples, did not show a uniform reduction of sulphur. This was due to the treatment of an insufficient quantity of ore to permit a proper regulation of feed, and consequently the exposure for too short a period of the mixture of ore and tar in the upper portion of the kiln where the roasting takes place.

The Empire Steel & Iron Co. propose to sink another shaft, so as to increase their monthly output to 16,000 tons and to concentrate all the ore mined, and to nodulize a fine ore carrying about 65% in iron. The concentration will incidentally reduce the phosphorus, as it occurs as apatite, and will probably reduce the sulphur from 3.00 to 1.00%, thus ensuring a nodulized magnetite, uniformly well below 0.10% in sulphur.

Negotiations for licences are pending with several companies owning large iron ore deposits or operating large blast-furnace plants, for although what has already been accomplished by this process has not yet been made generally public, large interests have nevertheless become convinced, from an examination of the plants already operating, and the success of the trial runs already made, that the process has solved the problem of the continuous economic nodulizing of fine ores or fine ferruginous by-products, accompanied by a continuous and uniform desulphurization of the high sulphur materials nodulized, to below 0.15%.

DESCRIPTION OF LABOR-SAVING DEVICES.

Several economic devices installed in the plants already operating or under construction are, on account of their ingenuity, worthy of notice.

At the Newark plant, dry bituminous coal is burned with a forced draft, the flame or flames entering the lower or discharge end of the kiln. Among the factors which cheapen the use of the pulverized coal is a coal dryer of special design and an automatic speeder and screw conveyor, with storage bin for the dry pulverized coal. With these devices the coal is used economically; an automatic and accurate record of the coal consumption is kept, and a perfect combustion maintained between the wide ranges of kiln temperatures used in nodulizing different materials. At the Pittsburg plant natural gas will be used; in the plants erecting or in contemplation, other fuels, solid, liquid, or gaseous, may be found more economical. The patented process is not confined to the use of any particular fuel or fuels to produce the proper range of temperature requisite for the successful application of the process to the nodulizing of fine materials, fritting at different temperatures.

An automatic plunger feed of very simple construction ensures a steady delivery into the kiln of any desired amount per hour of the material to be nodulized, independent of its fineness, or the amount of moisture it may contain. Uniformity in desulphurization can thus be assured by changing the rate of feed and the quantity of tar entering the kiln, when using materials containing different sulphur percentages, and also by regulating the speed at which the kiln is revolved.

A storage tank, of 150-ton capacity, into which the hot nodules are delivered from the kiln by a link belt conveyor is so arranged that the large steel trucks can be filled from the tank by the operation of a single lever, and if for any reason they are not at hand, the nodules, after the storage tank is full, are delivered on stock by an overhead shoot, into which the same link belt conveyor can also discharge.

LABOR REQUIRED FOR A TWO-KILN PLANT.

In the continuous operation of a two-kiln plant, with an output of at least 250 tons of nodules per 24 hours, for 26 days per month, the daily labor required is as follows:

	Day shift.	Night shift.
Boiler and engine:		
Engineer	1	1
Fireman	1	1
Kiln:		
Burner	1	1
Assistant burner	1	1
Helpers for oiling, etc.	2	2
Locomotive crane:		
Engineer	1	0
Fireman	1	0
General labor	1	1
Watchman and timekeeper	1	1
Chemist	1	0

A locomotive crane capable of handling 800 tons of material in 12 hours, and operating within a 25-ft. radius, can, on the day shift, keep a full day's supply in the storage bins, unload excess material on stock, and also shunt the trucks to and from the sidings, from which point they would be handled by the railway.

THE HENDERSON PROCESS FOR THE EXTRACTION OF COPPER.

In the application of this well-known process the National Metallurgic Co. have made a radical departure by adopting the rotary kiln at their Newark plant for the roasting of the ground mixture or copper pyrites cinder and salt. This will prove a great economy over the styles of furnaces formerly used, for the kiln and its accessories are automatic in their operation, and the labor, especially in handling and rabbling the mixture, is greatly reduced. The mixture is handled by a belt conveyor from the grinding mills to the storage tank, from which, by a bucket elevator, it is fed automatically through a screw feed into the kiln. Eventually the moist ore, free from copper, will be transferred by the overhead bucket crane to small tipping wagons that will be emptied into the storage tank, from which the ore will be delivered to the nodulizing kiln by the automatic plunger feed device, receiving its proper proportion of tar as it enters the kiln.

Copper pyrites cinder, containing as low as 1.50% of copper, can be handled with a profit in this modernized Henderson plant.

THE USE OF NODULES IN BLAST FURNACES.

Nodules made from pyrites cinder, concentrated magnetite, and flue dust, have been used, so far, at six different blast furnace plants, up to, in some cases, over one-half of the ore mixture, and in all cases with such satisfactory results that the demand far exceeds the present sources of supply.

The National Metallurgic Co. is about to erect its third plant, probably of four kilns, and is in active negotiation for putting up plants to convert fine sulphurous iron ores into marketable shape.

DESCRIPTION OF THE NODULIZING AND DESULPHURIZING PROCESS.

The invention consists mainly in purifying finely divided metalliferous materials, and forming them into nodules or lumps, by adding to the materials a binder, preferably reducing in character, adhesive at low temperatures (approximately below 600° F.), and volatile at fairly moderate temperatures (approximately 1,200° F.), which binder has an affinity for, and is capable of, forming volatile compounds with such impurities as sulphur, arsenic, etc. In practice, any adhesive hydrocarbon or carbohydrate compound, such as tar, pitch, asphalt, petroleum residuums, dextrine, molasses, glucose, etc., may be used provided it can be made liquid or plastic at comparatively low temperatures and can be readily and cheaply obtained.

The adhesive binder drops, or is sprayed, constantly into the stream of fine ferruginous material as it enters the upper end of an inclined rotary kiln; the material is therefore at once cohered into masses, which by the rotary action break up into nodules or lumps. One per cent. or less of an adhesive binder, such as tar, is sufficient. When the lumps reach a zone of higher temperature the tar combines with the sulphur and both are volatilized, the binder acting as a fuel as well. When the nodules approach the lower and hottest end of the kiln (approximately 2,000° F.) incipient fusion takes place, so that the product is a nodule, permanently cohered without the presence of any extraneous binder reducing the percentage of iron or other valuable metallic constituents contained in the original fine material.

As has been mentioned under the description of the plants, the process has already been successfully applied to a variety of fine materials fusing between a wide range of temperature, such as pyrites cinder, crude magnetite, magnetic concentrates, hæmatites (Mesaba), blast furnace flue-dust and franklinite (an iron-manganese-zinc ore).

NERNST LAMPS are made in five sizes for use on alternating current circuits of 110 or 220 volts and 25 to 133 cycles. They are adapted for either indoor or outdoor service and are listed as one-glower, two-glower, three-glower, four-glower and six-glower lamps. A one-glower lamp consumes 88 watts at 220 volts and is equivalent to three 16-c. p. incandescent electric lamps or one gas mantle. The two-glower lamp consumes 176 watts at 220 volts and is equivalent to seven 16-c. p incandescent electric lamps; the three-glower lamp consumes 264 watts at 220 volts and can be used to replace ten 16-c. p. incandescent electric lamps or one four-burner gas arc. A four-glower lamp consumes 352 watts at 220 volts and is equivalent to fourteen 16-c. p. incandescent electric lamps or one 6-amp. A. C. enclosed arc lamp. A six-glower lamp consumes 528 watts at 220 volts and is equivalent to twenty 16-c. p. incandescent electric lamps or one 7½-amp. A. C. arc lamp or one 5-amp. D. C. arc lamp. The above information was taken from a paper by Max Harris, read before the recent meeting of the Ohio Electric Light Association.

THE CUNARD STEAMSHIP "MAURETANIA," THE LARGEST VESSEL EVER BUILT.*

The "Mauretania," which was launched Sept. 20, is one of the two large steamships ordered by the Cunard Steamship Co. in fulfilment of its agreement with the British Government. She is of interest on account of her immense size and the fact that her steam turbine equipment is designed to give ten times the power of the largest marine turbine installation in use. Under the agreement with the British Government the Cunard Steamship Co. undertook to build for the British mail service two liners, to run at not less than 24½ knots in moderate weather, on the Atlantic passage, and to retain them at the call of the Admiralty for service in time of war. This agreement was not only acceptable to Great Britain on account of the advantage, or prestige, accruing from the possession of the fastest ocean passenger steamships afloat, but was welcomed by the marine engineering profession, as it promised the solution of important problems and conduced to further advances in propelling machinery. One of the ships was ordered from Messrs. John Brown & Co., Ltd., Sheffield and Clydebank, and the other, which forms the subject of the present article, from Messrs. Swan, Hunter, & Wigham Richardson, Ltd., Wallsend-on-Tyne, with machinery from the Wallsend Slipway & Engineering Co., Ltd.

The conditions of the contract are as follows:

> The vessels shall attain a speed of 25¼ knots on trial, and that within a year of going upon their station they shall maintain in one complete voyage to New York and back a mean speed of 24¾ knots.

Except in a few minor details, the "Mauretania" is a duplicate of the "Lusitania," which was launched some months ago; the turbines differ somewhat in construction, but their proportions are not dissimilar. The form of the propeller blades on the "Mauretania" is more elliptical, and the hull is of 6-in. greater depth, which increases her gross tonnage by 700 tons. The dimensions of the "Mauretania" are:

Length over all	785 ft.
Length between perpendiculars	760 ft.
Beam, extreme	88 ft.
Depth, molded	60 ft. 6 ins.
Gross tonnage	33,200 tons.
Maximum draft	38 ft.
Displacement at maximum draft	43,000 tons.

Figs. 1 and 2 illustrating the longitudinal section and plan of the ship, show the arrangement of the decks and bulkheads. Excluding the hold, there are nine decks. These are: the lower orlop, the orlop, the lower, the main, the upper, the shelter, the promenade, the boat and the sun decks. In the center of the ship, however, there are only six decks, the boiler, machinery and coal-bunkers occupying fully 420 ft. of the total length of the vessel, and the depth from the main deck to the hold. There is very little capacity for cargo, the vessel being exclusively intended for carrying passengers and mails.

Two electric passenger elevators in the center of the ship run to each of the six passenger decks. Instead of the steel decking of the ship being covered with wood, the floors are lined with corticine, a special cork cement extensively used in warships to minimize fire risk, and in this case largely to save weight.

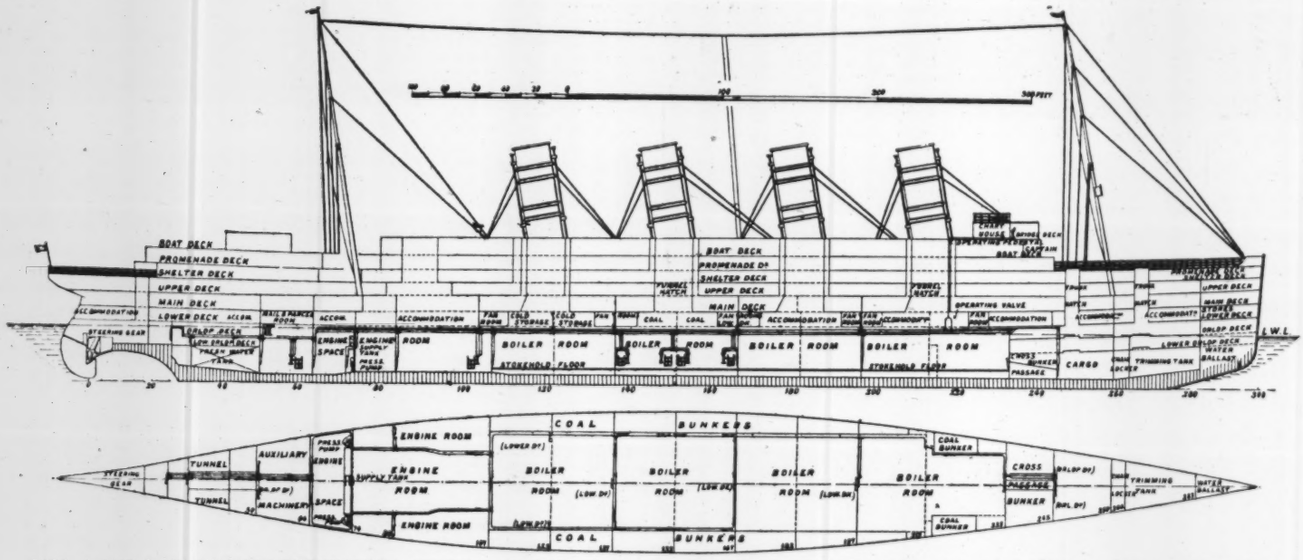
The numbers of passengers and crew provided for are:

First-class passengers	560
Second-class passengers	500
Third-class passengers	1,200
Crew	810
Total population	3,070

The decks with state-rooms are for the most part 9 ft. in height, but where there are saloons the height is from 10 to 11 ft. The top of the wheel-house is 100 ft. above the keel, and the funnels 152 ft.

There are 15 transverse bulkheads closely spaced forward and aft, where there is greater chance of injury owing to collision. Two of the largest of the compartments might easily be flooded without any difficulty arising. The coal-bunkers are also fitted with bulkheads, and as shown in Figs. 1 and 2 there is a well-arranged scheme for closing all the doors simultaneously on the Stone-Lloyd system. In the event of any contingency, every door can be closed by hydraulic power by the officer on the bridge, or at one of several positions throughout the ship. A door then can only be reopened from within the compartment to liberate any one imprisoned, and even in such case the door closes again automatically after the lapse of a sufficient period of time to allow any one to pass through.

The construction of the double-bottom of the ship is interesting. The flat keel is made up of three thicknesses of plating, varying, according to the position in the ship, from 1 in. to 1¼ ins. thick. The holes were, in all cases, formed by electric drills, with the plates superposed, so as to secure absolute alinement, and the riveting was done by hydraulic power. This, indeed, was used throughout the whole of the double bottom, and as the plates were for the most part 1 in. to $1^1/_{16}$ ins. thick, the conditions were more analogous to boiler making than ordinary ship riveting. The depth between the outer and inner bottom is, at the center, 5 ft.



FIGS. 1 AND 2. PLAN AND LONGITUDINAL SECTION OF THE QUADRUPLE-SCREW TURBINE-DRIVEN 25-KNOT CUNARD LINER "MAURETANIA."

On account of the high speed, the entry, as shown in Fig. 3, is exceptionally fine. As is indicated by this view, there is not in any part a hollow; rising from a sweet curve from the keel which extends almost to the vertical stem, the lines are straight until slightly above the water line, where they flare out. Wedge-shaped bars are fitted along the laps of the shell plating so as to obviate any possibility of the anchor catching in the projection formed by the laps. A peculiar form of anchor hawse-pipe is used; it is thought this form will enable the anchor to be more snugly housed, and thus reduce the disturbance due to waves when the vessel is running at high speed through a heavy sea.

The amidship framing is of channel bars with deep web frames at intervals, which vary according to their position in the ship. They are most frequent and heaviest, of course, at the machinery spaces. The channel bars extend from the margin plate of the double bottom, to which they are connected by heavy brackets, to the sheltered deck, 60 ft. 6 ins. above the level of the keel. The length of the bars is 55 ft., and they weigh in most cases 15 cwt.

The rudder weighs 65 tons, and is of the balanced type, made up of a series of castings, bolted together with heavy ribs. The rudder-head is of forged ingot steel, of 25½-in. diameter.

There are four turbine shafts, each with one propeller. The two outer shafts are driven by high-pressure turbines, while on the two inner shafts are the low-pressure turbines, with the astern turbines located at the after end. The condensers are placed to the rear of the engines, and outside of the inner shafts. The two inner shafts turn outward and the two outer shafts inward. As a result of experiment, it was considered that this arrangement would give better results than with all propellers working in the same direction. The total power will be 68,000 I. HP., equally divided through the four shafts, which will run at from 190 to 200 r. p. m.

Fig. 4 shows one-half of the turbine installation. The group illustrated has a length of 150 ft., and the height from the floor is over 20 ft. The turbine bearings have been given a spherical form to afford greater freedom. In the seats there are adjusting pieces for taking up the wear at the sides and bottom, so that the bearings can be reset without the shaft being lifted for the renewing of the white metal.

The rotor disks differ from general practice. They are dished out, forming a triangle in elevation. This should add to their rigidity—an important desideratum in view of the tendency toward a whipping action by a rotor of 130 tons, nearly 12 ft. in diameter, making 200 r. p. m.

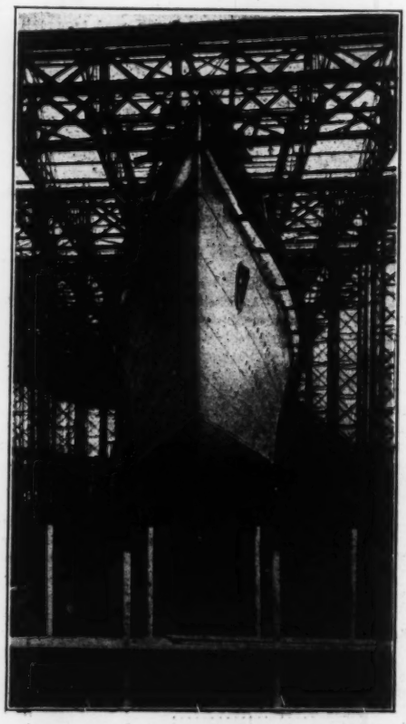


Fig. 3. The Bow of the Ship Showing the Fine Entry for High Speed.

*Abstract of an article in "Engineering."

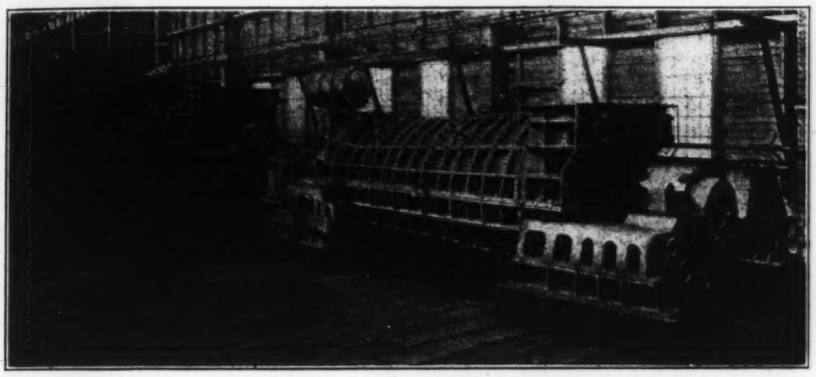


FIG. 4. A VIEW OF THE TURBINES, THE HIGH-PRESSURE TURBINES BEING IN THE FOREGROUND.

The shafting, of Whitworth steel, is, in the case of the low-pressure turbines, about 33 ins. in diameter, and in the high-pressure turbines 27 ins., slightly increased near the glands; but the line shafting is 20 ins.; in all cases the shafts are hollow.

The steam installation includes 23 double-ended and two single-ended boilers, the former having eight, and the latter four, furnaces, so that there are 192 furnaces in all. The larger boilers are 17 ft. 3 ins. in diameter and 21 ft. long. The boilers are to work under a forced draft system, and the fans will be driven by electric motors. There will be about 160,000 sq. ft. of heating surface and nearly 4,000 sq. ft. of grate area. The working pressure at the boilers will be 180 lbs., and at the turbines 160 lbs. The boilers will be arranged in four stokeholds, with seven boilers in the forward stokehold, and six in each of the others. For each of the four groups of boilers there will be a funnel which will rise to a height of 152 ft. above the keel of the ship. These funnels are elliptical in plan, and measure externally 23 ft. 6 ins. × 17 ft. 6 ins.

A CROWDED EXPRESS TRAIN on the Great Northern Ry. was wrecked outside of Grantham, England, at midnight Sept. 19, killing 10 persons and injuring 16 others. The wreck was a repetition of the Salisbury accident on the London & Southwestern Ry., July 1, 1906 (Eng. News, July 5, 1906, p. 24). The train known as the "Scotch Express" should have stopped at Grantham but instead passed through the city at high speed. Shortly after passing the station the train left the track on a curve, smashed the parapet of a bridge, the engine and several coaches rolling down an embankment. The coaches immediately took fire and burned until 5 o'clock in the morning.

A CARLOAD OF DYNAMITE, which exploded in the yards of the Louisville & Nashville R. R., at Jellico, Tenn., Sept. 21, killed 12 persons and injured scores of others, besides destroying six buildings and badly damaging others. Two explanations are given for the explosion, one that the car was hit by another car loaded with pig iron, the impact causing the explosion, and the other that some people were shooting at a mark on the car and that a bullet exploded the dynamite. The car contained 20,000 lbs. of the explosive. The property loss is placed at from $500,000 to $1,000,000 by press reports.

A TYPHOON swept over the harbor of Hongkong Sept. 18 causing great damage to property ashore, as well as to shipping. The cyclonic storm struck the town at 10 a. m. and lasted two hours, sinking in that time some 12 ships, while some 24 others were driven ashore. This does not include the native craft, large numbers of which were destroyed. The list of wrecked ships includes a British gunboat and a French torpedo boat destroyer. The loss of life is variously placed by press dispatches from 100 to 10,000, mostly natives, while the property loss is estimated at more than $1,000,000.

THE ROOSEVELT DAM in Arizona was officially begun by the formal laying of the first stone by the contractor on Sept. 20. The contract for the dam was signed April 21, 1905, but a series of unusual floods in the Salt River has greatly delayed the work of excavating to bed rock for the foundations. It is expected to have the dam raised to a point where some of the stored water will be available for irrigation in 1907. The dam will be 284 ft. high, 280 ft. long at the base and 700 ft. long on top. It will back the water up for about 25 miles, forming a lake with a capacity of 1,300,000 acre-feet.

THE TWIN TUNNELS OF THE HUDSON CO., connecting Jersey City with New York under the North River, are about to receive their electric equipment, which is to be furnished by the General Electric Co. Fifty electric cars will be operated, taking their power from the third rail. Each tunnel will have a single track; the north tube carrying the west-bound traffic and the south tube carrying the east-bound traffic. The cars will be operated in trains by the Sprague General Electric system of multiple-unit control in a manner similar to that employed on the New York subway trains. Each car will be equipped with two 100-HP. railway motors. Power for this new development will be supplied from a large station on the New Jersey side, located between Jersey City and Newark. Curtis steam turbines will be employed. The initial equipment includes two 3,000-KW., 11,000-volt machines and two 6,000-KW., 11,000-volt machines. The total power generated will be distributed at high voltage to three substations where the alternating current will be stepped down to 650 volts direct-current through transformers and rotary converters.

FIG. 5. THE STERN OF THE SHIP SHOWING THE POSITION OF THE PROPELLERS AND RUDDER.

THREE LARGE SHIPS were launched, at as many different shipyards, in England, Sept. 20. The new Cunard steamship "Mauritania" was launched at the yards of Swan, Hunter & Wigham Bros., at Wallsend-on-Tyne. This is a sister ship to the "Lusitania" (Eng. News, Aug. 23, 1906, p. 189), which was launched on the Clyde last June. The "Mauritania" is 790 ft. long, with an 88-ft. beam and a gross tonnage of 33,200 tons, and will be driven by 70,000-HP. steam turbines at a speed of 25 knots. She will have accommodations for 500 first-class, 500 second-class and 1,300 third-class passengers and a crew of 800. The White Star liner "Adriatic" was launched at the yards of Harland & Wolff, at Belfast. She is 725 ft. long with a gross tonnage of 25,000 tons, and will have accommodations for 3,000 passengers and a crew of 350. The new British cruiser "Shannon" was launched at Chatham. She is 490 ft. long, with a draft of 25 ft., a 75½-ft. beam and a tonnage of 14,500 tons. Her engines will develop 27,000 HP. and a speed of 23 knots. She will be equipped with four 9.2-in. barbette guns, ten of 7.5-in. caliber, thirty-two quick-firers and five torpedo tubes. Her crew will number 755 men.

THE SALTON SEA, in Lower California, is reported to be still rising, and to have attained such a height that the Southern Pacific R. R. is again raising a large mileage of track to obviate flooding. The breaking of the Colorado River into this below-sea-level area was described and discussed at some length in our issues of Feb. 22 (p. 210), March 15 (p. 300), May 10 (p. 512), and June 28 (p. 721), 1906.

THE ERECTION OF THE MARYLAND AND VIRginia Buildings for the Jamestown Exposition in Norfolk, Va., was commenced Sept. 19 by the laying of the respective cornerstones. Ground was broken at the same time for the Missouri building and the headquarters of the Travelers' Protective Association. Very elaborate ceremonies commemorated the occasion, Governor Swanson, of Virginia, Governor Warfield, of Maryland, a distinguished gathering of officials and representatives of the two commonwealths, officers of the army and navy, and many residents of the surrounding country being present. This move marks the beginning of an exposition which will recall to Americans where the United States was born.

A TUNNEL 6¼ MILES LONG is contemplated by the Denver, Northwestern & Pacific Ry. (the Moffat road) in place of the two-mile tunnel originally planned (Eng. News, Mar. 9, 1905, p. 241). The proposed tunnel would enter the mountain 2½ miles beyond Tolland, Col., at an elevation of 9,000 ft., thereby saving a climb of 900 ft. and a distance of 14 miles over the proposed two-mile tunnel. A third tunnel is being considered by the engineers, which would be approximately 4 miles long and located at an elevation of about 9,300 ft. The cost of boring the six-mile tunnel is roughly estimated at from $4,000,000 to $5,000,000 and the time of construction about three years.

FIVE SURVEYS are being made by the Astronomical Branch of the Department of Interior of Canada, four of them being in conjunction with the Coast and Geodetic Survey of the United States. One party is engaged in making an astronomical survey of the 141st meridian of west longitude, which is the boundary line between Canada and Alaska from Mt. St. Elias to the Arctic Ocean. A second party is running the line known as the "Award Boundary," which was defined by the London award as the dividing line between Alaska from Mt. St. Elias to the Portland Canal. The third survey consists of replacing the monuments that mark the 49th parallel of north latitude, this being the southern boundary of British Columbia. A fourth expedition is doing similar work on the boundary line between Quebec and Vermont. The fifth party, entirely Canadian, is making a trigonometrical survey between Ottawa and the St. Lawrence River in order to establish an accurate basis for future work.

WATER SUPPLY AND FIRE PROTECTION at Springfield, Mass., has been reported on by the Committee on Fire Prevention of the National Board of Fire Underwriters (W. E. Mallalieu, Assistant to General Agent, 135 William St., New York). The gravity supply mains are characterized as "too small to maintain pressure at times of maximum consumption, thus emphasizing the necessity for haste in construction of new works." The pressures are generally satisfactory, except that they are not

well maintained in the higher parts of the city. The spacing of the gate valves is too great, as also some of the fire hydrants in the congested value district. The supervision of the fire department is considered unsatisfactory, and the fire engines too few. The fire alarm system is also unsatisfactory. The building laws are inadequate and poorly enforced. There is practically no municipal control of electrical wiring and allied risks. The conflagration hazard is described as follows:

Severe, owing to narrow streets, the lack of horizontal and vertical opening protection, numerous serious exposures, hazardous centers of high combustibility, grouping of bad blocks and a fire department inadequate for coping with large fires. A mitigating feature is found in the fact that a number of the more hazardous risks are sprinklered, but the resultant conflagration hazard is still considerable.

THE REGULAR WATER SUPPLY OF ROGERS Park, now a portion but formerly a suburb of Chicago, has been cut off by the city, and connections made with the city mains. It is alleged by the city authorities that the supply from the intake of the Rogers Park Water-Works Co. is badly polluted, as was explained at some length in a note published in our issues of Aug. 16 and 30, 1906, pp. 179 and 289. The water company alleges that the action on the part of the city authorities is a political move, and that it is also intended to diminish the purchase price of the works in case they are taken over by the city. There has long been friction between the city and the company over the water rates charged by the latter.

ANOTHER FERRIS WHEEL is to be demolished, its operation having become unprofitable. The wheel is at Earl's Park, London, a well-known amusement resort. The original Ferris wheel, which had a brief period of renewed popularity at the St. Louis Exposition, two years ago, was demolished last year.

GOVERNMENT OWNERSHIP OF RAILWAYS IN Japan was provided for by the last Japanese diet. The government of that country has since announced that railways with the following mileage will be bought before the end of the present year: Nippon Ry., 861 miles; Sanyo Ry., 406 miles; Kobu Ry., 28 miles; Hokkaido Colliery Ry., 208 miles; Nishinari Ry., 4 miles; Ganyetsu Ry., 49 miles; total, 1,556 miles. The total purchase price for the 1,556 miles is to be $123,500,883, which will be met by a 5% bond issue. [U. S. Consular Reports, Sept. 14, 1906.]

MACHINERY COMPETITION in export trade between the United States and Great Britain is given attention in the "Daily Consular and Trade Reports" for Sept. 13, 1906. Both these countries are rapidly increasing their foreign sales, the American statistics for the fiscal year ending June 30, 1906, showing $93,448,397, or an increase of 18⅛% over 1904, and the British figures for the first six months of 1906 showing $64,836,326, or 24½% increase over the same months of 1904. In locomotives the United States exports increased 20% from 1904 to 1906, while British exports increased 35%. The most notable American increase was in the Central American States, where $37,150 worth of locomotives were sent in 1904, $60,810 in 1905, and $1,131,930 in 1906. South American markets proved the best field for British locomotive expansion, the sales there increasing from $785,000 for the first half of 1904 to $3,180,000 for the first half of 1906. In agricultural engines the United Kingdom holds the heavy end of the foreign trade, and in other engines is strengthening her sales position. The exports amounted to $9,068,169 for the first half of this year, against $5,452,889 in the same period of 1904. The South American sales of British stationary engines advanced from $500,000 in the first half of 1904 to $1,200,000 for the first half of 1906. In agricultural machinery and tools the American exports amounted to $24,554,427 in 1906 against $22,749,700 in 1904, and the British exports for these respective years were $2,904,891 and $2,582,068. In pumping machinery Great Britain does not compete with America, which in 1906 exported $4,210,624 worth against $2,703,397 in 1904. In electrical machinery America has a long leadership through the highest type of motor and other electrical appliances. The domestic sale of these is enormous, but the surplus for foreign markets went up from $5,645,809 in the fiscal year 1904 to $7,869,137, fiscal year 1906. Canada proved the best customer, taking last year $2,138,134 worth, and the United Kingdom $1,153,528 values. Cuba, which purchased only $31,786 worth of American electrical machinery in 1904, last year took $528,565 worth. The exports to Japan, amounting to $726,492 for 1906, were somewhat less than for the two previous years. But that Great Britain was not idle in the electrical field is shown by the doubling of exports in this machinery in comparing the first half of 1906 with the same period of 1904.

FOREST PLANTING ON COAL LANDS is proposed by the H. C. Frick Coke Co. This company, with the co-operation of the Forest Service Division of the U. S. Department of Agriculture, has selected a number of sites for planting, in the vicinity of Scottdale, Pa. The land is at an elevation of about 1,200 ft. above sea level. A planting plan has been made for about 456 acres of unproductive agricultural land and of land the topography of which has been modified by settlements due to mining. The following is a summary of the species and acreage recommended: Red oak, 110 acres; red oak and chestnut, 122; red oak and hard maple, 79; red oak and European larch, 45; red oak and yellow poplar (tulip-tree), 12; chestnut oak (rock oak), 24; white oak, 31; European larch, 25; yellow poplar, 5; hardy catalpa (western catalpa), 3; total, 456 acres. The foregoing has been taken from Forest Service Circular No. 41, U. S. Department of Agriculture, by S. N. Spring, Forest Assistant.

THE HYDRO-ELECTRIC PLANT AT KAKABEKA Falls, Ont., has just been completed and put into service. It has a capacity of 10,000 HP. and supplies power to Fort William, a distance of 16 miles from the Falls. The waters of the river are diverted in a 10-ft. pipe at a point ¾-mi. above the Falls. The pipe is led some distance below the Falls to a reservoir, from which there is a drop of 180 ft. through a steel penstock to the power house below. Cement and steel have been largely used in the construction. One of the special features in the design of the plant was the disposal of the anchor ice, which has been the source of a great amount of trouble to water plants in that part of the country. The power is transmitted at 25,000 volts to a substation at Fort William. Arrangements have been made for increasing the initial output from time to time as required, in increments of 10,000 HP. The work cost $2,000,000 and has been supervised by Mr. R. W. Leonard. Associated with him were Mr. R. S. Kelsch, Elect. Engr., and Mr. William Kennedy, Jr., Hydraulic Engr., both of Montreal.

A LARGE LAUNDRY is to be installed at Chicago, Ill., by the Illinois Central R. R. for cleaning all the linen used on the company's lines. The laundry will be designed to wash and iron 90,000 napkins, 30,000 tablecloths, 10,000 aprons, 6,000 jackets, 50,000 towels, and 2,000 caps, table felts, etc. per month.

PERSONALS.

Mr. R. I. Parks has been elected City and Sewer Engineer of Denison, Tex.

Mr. L. N. Hinman, of Sheridan, Wyo., has been appointed Civil Engineer at Boise Barracks, Idaho.

Mr. C. V. Taylor, Assistant Engineer, and Mr. Geo. G. Stroebe, Engineering Aids, in the Reclamation Service, have resigned.

Mr. Jas. F. Parr has resigned as Assistant Engineer of the Mississippi River & Bonne Terre Ry. to become Assistant Engineer of East St. Louis, Ill.

Mr. George Rommel, Jr., who lately resigned as Engineer in Charge of Sewers, of Wilmington, Del., has accepted the position of Consulting Engineer of Pensacola, Fla.

Mr. J. N. Kennedy, Superintendent of the Alma Cement Co., of Wellston, Ohio, has resigned to become associated with Mr. B. B. Lathbury, Consulting Engineer, Philadelphia, Pa.

Mr. James L. Sharkey has been appointed Sales Manager in the Department of Expanded Metal, Metal Lath, Twisted Lug Bars and Girder Frames, of the New York office of the General Fireproofing Co., of Youngstown, Ohio.

Mr. R. J. Mansfield, Jun. M. Am. Soc. C. E., formerly of the Engineering Department of the Wellman-Seaver-Morgan Co., of Cleveland, Ohio, has gone to Manila, P. I., in the interest of the Atlantic, Gulf & Pacific Co., Contractors, New York City.

Mr. H. M. Hallock, Superintendent of the Panhandle Division of the Chicago, Rock Island & Pacific Ry., has been made General Superintendent of the Choctaw Division, with headquarters at Little Rock, Ark. He succeeds Mr. W. S. Tinsman, who has been transferred to Topeka, Kan.

Mr. Peter W. Hoffman, formerly employed by Siemens & Halske, has been appointed by the Canadian Government Commission (Hydro Electric Power Co.) Expert Engineer to receive and report on the applications for municipal power and to have general management of plant construction.

Mr. H. A. Dewey, member of the Railway Commission Staff of Canada, has been appointed Engineer for Western Canada, with headquarters at Winnipeg. He will have charge of railway matters from that part of Canada coming before the Commission, investigate railway accidents, etc.

Mr. Charles E. Magoon, Governor of the Panama Canal Zone, has been appointed Vice-Governor-General of the Philippines. Gov. Magoon's successor has not as yet been named, and it is reported that the office of governor of the Canal Zone may be discontinued and its duties transferred to Chairman Shonts of the Isthmian Canal Commission, or Chief Engineer Stevens.

Mr. George W. Jackson, Chief Engineer and General Manager of the Construction Department of the Illinois Tunnel Co., has resigned, the tunnel having been completed. After a tour of Europe Mr. Jackson will devote his time to directing the affairs of the Jackson & Corbett Steel & Iron Co., the Jackson & Corbett Co., and the Jackson Steel Sheeting Co., of which companies he is President.

Mr. R. B. Hall, Jr., formerly Engineer for Mackenzie, Quarrier & Ferguson, of New York City, and Mr. Illges, late of the engineering staff of the General Electric Co., have formed a partnership under the firm name of The Hall-Illges Co., Engineers, Contractors, Manufacturer Agents. They have opened offices in the Candler Building, Atlanta, Ga., where they will engage in the design and installation of high pressure steam and electric plants, etc.

Obituary.

William A. Field, President of the Hesperia Land & Water Co., of Los Angeles, Cal., died at his home in that city Sept. 9.

J. C. Goersch, an architect of Los Angeles, Cal., committed suicide in a canyon near that city by shooting himself with a rifle, Sept. 18.

John R. Pearson, formerly Vice-President and General Manager of the Indianapolis Gas Co., died at his home in Indianapolis, Ind., Sept. 19, of apoplexy.

Philip F. Kobbe, Vice-President and Treasurer of the Westinghouse Electric & Mfg. Co., died at his country place, Hillfield, Stockbridge, Mass., Sept. 21, aged 64 years. He was born in New York City and was educated in Germany. Mr. Kobbe was formerly connected with the United States Lighting Co., but when that company was absorbed by the Westinghouse company he was made Vice-President and Treasurer of the later corporation.

James Abercrombie Burden, Sr., President of the Burden Iron Works, of Troy, N. Y., died at his home in New York City, Sept. 23, of gastritis. Mr. Burden was born at Troy, N. Y., Jan. 6, 1838, and was educated under a tutor, supplemented by lectures at Yale Scientific School and Rensselaer Polytechnic Institute. He then entered his father's iron works and his active business life was spent in their management, and in connection with interests associated with them. He made a number of inventions of some importance. Mr. Burden was a member of the American Society of Civil Engineers, the American Society of Mechanical Engineers, and the American Institute of Mining Engineers, and also joined several foreign scientific societies.

ENGINEERING SOCIETIES.

COMING MEETINGS.

LEAGUE OF AMERICAN MUNICIPALITIES.
Sept. 26 to 28. Annual convention at Chicago, Ill. Secy., Jno. MacVicar, Des Moines, Iowa.

AMERICAN ELECTROCHEMICAL SOCIETY.
Oct. 8-9. Annual meeting at New York City. Secy., S. S. Sadtler, 39 South 10th St., Philadelphia, Pa.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 9 to 12. Annual Meeting at Birmingham, Ala. Secy., G. W. Tillson, Municipal Bldg., Brooklyn, N. Y.

AMERCAN STREET AND INTERURBAN RAILWAY ASSOCIATION.
Oct. 15 to 19. Annual Convention at Columbus, Ohio. Secy., Bernard V. Swenson, 60 Wall St., New York City.

ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS.
Oct. 16. Annuel Meeting at Boston, Mass. Secy., S. F. Patterson, Concord, N. H., c/o Boston & Maine R. R.

THE RAILWAY SIGNAL ASSOCIATION.
Oct. 16-18. Annual Meeting at Washington, D. C. Secy., H. S. Balliet, Grand Central Station, New York City.

AMERICAN GAS INSTITUTE.
Oct. 17-19. First Annual Meeting at Chicago, Ill. Acting Secy., James W. Dunbar, New Albany, Ind.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Nov. 13-15. Annual Meeting at Chicago, Ill. Secy., C. E. Jones, Beardstown, Ill.

AMERICAN MINING CONGRESS.
Nov. 13-17. Annual Meeting at Denver, Col. Secy., J. F. Callbreath, Jr., Denver, Colo.

AMERICAN PUBLIC WORKS ASSOCIATION.—The second annual meeting of the association was held at Atlanta, Ga., Sept. 14-15, at which time the association's rules regarding contracts were revised and the following officers elected: President, Mr. R. C. Huston; Vice-Presidents, Mr. F. Cole, Mr. Chas. G. Wilson and Mr. Geo. O. Tenny; Secretary, W. H. Flint; Treasurer, Mr. Morgan Llewellyn; Chairman Executive Committee, Mr Edward J. O'Beirne.